DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH

ROAD RESEARCH LABORATORY

SOIL MECHANICS
FOR
ROAD ENGINEERS

LONDON

HER MAJESTY'S STATIONERY OFFICE

1952 : Reprinted 1961

FOREWORD

DURING the war, lecture courses in the advanced techniques of road and airfield construction were arranged at the Road Research Laboratory for personnel of the Army and the R.A.F., the lectures being subsequently published in book form under the title " Soils, Concrete and Bituminous Materials." As a result of requests from many sources a series of lecture courses for civilian engineers, dealing mainly with road techniques and emphasizing normal peacetime requirements, was begun in 1946 and has since been a regular feature of the Laboratory's work.

Since 1945, when " Soils, Concrete and Bituminous Materials " was published, many advances have been made in knowledge of the principles governing the technique of road construction, and year by year the lecture courses have been revised to keep pace with developments. These changes, and the fact that the emphasis of the original publication was on short-term military requirements, have created the need for a new publication which can be used both as a textbook for the lecture courses and as an up-to-date work of general reference.

The present book is the first of three volumes which will replace the original textbook, the change from the single volume having been made to accommodate the greater amount of material now available, and to suit the convenience of the reader who may be concerned with only one of the three subjects. The remaining two volumes, dealing with concrete and bituminous roads, will be issued in due course.

Most of the chapters in the present volume are based on the lectures delivered at the courses by the members of the staff of the soil mechanics section of the Laboratory who are named overleaf. The subject matter of these chapters has been edited as far as was needed to avoid wide differences of treatment, but it has been thought unnecessary, and indeed undesirable, to impress a uniform standard of style. Four chapters have been added to round-off the volume and cover certain aspects of the subject not dealt with in the lectures. The editorial work was undertaken by Mr D. J. Maclean, the head of the Laboratory's soil mechanics section, assisted by Miss M. G. Greysmith and Dr T. J. Lonsdale.

The main sources of the information in the book have been the Laboratory's own researches into soil mechanics. But this information has been supplemented and broadened by a continuous study of technical literature from world sources, by the lessons learned from examining constructional problems all over Great Britain and in the Colonies and, above all, by the very close personal contacts maintained by the research workers with practising engineers employed by highway authorities and contractors. The book attempts, therefore, to present the collective experience of a large body of people concerned with the use of soils and soil engineering techniques in road construction and maintenance.

It is hoped that the book will be useful to many engineers in Great Britain and the Commonwealth besides those who actually attend the Laboratory's lecture courses, and that it will be useful also to engineers concerned with airfield construction and with other operations involving techniques similar to those applied to roads.

Most of the material in this book was written prior to December, 1949 but alterations were subsequently made in 1950 and 1951 to bring the book in line with recently published Codes of Practice, British Standards and other similar publications.

In this country, as well as overseas, rapid progress is being made in the use of soil as an engineering material, and it is likely that an early revision of some chapters of the book will be necessary. Suggestions and criticisms from readers will therefore be particularly welcome and will be carefully considered.

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1st August, 1951.

ACKNOWLEDGEMENTS

The following members of the soil mechanics section of the Road Research Laboratory collaborated in preparing and giving the lectures forming the substance of this book: K. E. Clare, D. Croney, E. H. Davis, W. A. Lewis, D. J. Maclean, H. W. W. Pollitt, F. A. Shergold, J. O. Tresidder, S. B. Webb and F. H. P. Williams.

Acknowledgements are due to Dr E. W. Russell of the Department of Agriculture, University of Oxford, and Dr A. W. Skempton of the Imperial College of Science and Technology, City and Guilds College, London, for their valuable criticisms of the text of Chapter 2 and Chapter 19 respectively.

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METRIC EQUIVALENTS OF BRITISH UNITS

1 in.	=	25·4 mm.
1 ft	=	30·48 cm.
1 sq.in.	=	645·2 sq.mm.
1 sq.yd	=	0·836 sq.m.
1 cu.in.	=	16·4 cc.
1 cu.yd	=	0·764 cu.m.
1 lb.	=	0·454 kgm
1 ton	=	1,016·0 kgm
1 lb./in.	=	17·86 kgm/m.
1 lb./sq.in.	=	0·0703 kgm/sq.cm.
1 lb./cu.ft	=	16·02 kgm/cu.m.
1 lb./sq.in./in.	=	0·0277 kgm/sq.cm./cm.
1 ton/sq.ft	=	1·094 kgm/sq.cm.
1 H.P. (550 ft lb./sec.)	=	76·04 m.kgm/sec.
t°F.	=	$\frac{9}{5}$ T°C. + 32

BRITISH EQUIVALENTS OF METRIC UNITS

1 mm.	=	0·0394 in.
1 cm.	=	0·3937 in.
1 m.	=	1·094 yd
1 gm	=	0·035 oz.
1 kgm	=	2·2 lb.
1 ml.	=	0·061 cu.in.
1 gm/cc.	=	62·43 lb./cu.ft
T°C.	=	$\frac{5}{9}$ (t°F. − 32)

CHAPTER 1

SCOPE OF THE BOOK

INTRODUCTION

1·1 The materials which compose the crust of the earth may be divided broadly into two main groups, rock and soil, the latter being taken to include all deposits of loose material. An adequate knowledge of the properties of these materials is essential for the proper design and construction of roads and airfields, since they not only form the foundation on which these structures are constructed but also are the principal materials used for their construction. It is thus necessary in road and airfield engineering to understand the properties of rock and soil in relation to their behaviour both as foundation materials and as construction materials; this book is an account of the present state of knowledge of this subject.

1·2 This chapter indicates briefly the scope of the subject, and in the following chapters the various aspects are treated in more detail on the basis of information obtained from investigations made at the Road Research Laboratory supplemented by information derived from a study of the literature of the subject.

ROAD AND AIRFIELD STRUCTURES

1·3 The main structural elements involved in road construction are dealt with in the following order:—

(1) The embankment.
(2) The foundation beneath the embankment.
(3) The cutting.
(4) The road pavement.

1·4 Although cutting and filling are required in the construction of most airfields, they are essentially composed of a single structure, the airfield pavement, which corresponds closely to the road pavement but which has usually to support greater loads. There is thus no fundamental difference between roads and airfields, and most of the information contained in this book is of equal application to airfields.

THE EMBANKMENT

Structural Problems

1·5 A road embankment, if properly designed and constructed, should possess stable slopes and should not settle to any great extent.

1·6 STABILITY OF SLOPES. In its worst form the slip of an embankment slope may result in the complete destruction of a length of road. Fortunately this is of rare occurrence, but minor slips, in which the soil foundation of the road moves outwards and downwards to a small extent, are quite common. These frequently occur where the road has been constructed too close to the edge of

the embankment, particularly as a result of the widening of the road. With concrete roads this usually results in the opening of the central longitudinal joint as well as in the settlement of slabs.

1·7 There are the following types of slip:—

(1) The sliding of one stratum on another (in the case of embankments constructed on sidelong ground).
(2) Rotational slips, in which a mass of soil slips on an interface in the form of an arc of a circle.
(3) Surface slips, which consist of the movement of a thin layer of material down the surface of the slope.

Rotational slips have gained considerable prominence in soil mechanics owing to their susceptibility to mathematical analysis. Experience has shown that they are not unduly common on roads in this country. The occurrence of slips often indicates the need for drains for intercepting seepage water. Chapter 26 is devoted to the problem of the stability of clay slopes.

1·8 SETTLEMENT OF EMBANKMENTS. The settlement of embankments results from:—

(1) The compression of the embankment filling.
(2) The consolidation of the foundation of the embankment.

The second factor will be discussed later in this chapter.

1·9 The compression of the embankment filling is caused by the expulsion of air from the material by the weight of the pavement and the higher soil layers. This can be reduced to negligible proportions by adequate compaction of the fill during construction. It is particularly important to prevent large differential settlement occurring over a short length as, apart from its unsightliness, damage may be caused to the road, particularly when constructed of concrete. Differential settlement is most liable to occur where there is a sudden change in the thickness of fill, e.g., at an underbridge, or where the compaction of the fill is particularly difficult, e.g., at bridge abutments.

1·10 The settlement of embankments is dealt with in more detail in Chapter 24.

Construction Problems

1·11 The principal problems involved in the construction of road embankments are the choice of the filling and its compaction into a dense, stable mass.

1·12 FILL MATERIAL. The technical requirements of the fill material are first that it should provide, when properly compacted, a stable structure free from settlement and second that it should not deteriorate to any great extent from the action of weather.

1·13 The first requirement rules out the use of peat and organic clays for constructing embankments. A rough guide to the quality of a material for filling is its maximum dry density as determined by the standard compaction test (B.S. 1377).

1·14 The principal materials that are susceptible to weathering are certain industrial wastes which are attractive on account of their low cost.

1·15 COMPACTION. The adequate compaction of the fill material is particularly desirable in the construction of earthworks. It reduces settlement, increases the stability of the slopes and reduces the tendency of the material to absorb water. Considerable advances have been made in recent years in our knowledge of soil compaction, and considerable effort has been devoted to its study by the Road Research Laboratory. The compaction of soil is fully dealt with in Chapter 9.

THE FOUNDATION BENEATH THE EMBANKMENT

Structural Problems

1·16 Embankments require a stable foundation in the same way as other structures. When the underlying soil consists of peat or soft clay, the weight of the embankment causes settlement due either to the consolidation of the foundation soil or, in extreme cases, to the ultimate failure of the soil, in which a sudden settlement of the embankment is accompanied by the heaving of the ground at some distance from the toe of the embankment.

1·17 CONSOLIDATION OF FOUNDATIONS. Consolidation is the process in which the soil is compressed under load, usually by the expulsion of water from the pores of the soil. It is possible by the analysis of the results of a special laboratory test on the foundation soil to make a fairly accurate estimate of the probable settlement and the time required for it to occur. This subject is considered in detail in Chapter 23.

1·18 Although it is not possible to prevent settlement occurring in the foundations of embankments, the period during which it occurs may be reduced sufficiently to enable movements to be completed before the final road surfacing is constructed. This is achieved by the insertion of vertical sand drains in the foundation; these have the effect of reducing the path, and hence the time taken, for water to escape from the pores of the soil. Further details of this method are given in Chapter 23.

1·19 BEARING CAPACITY OF FOUNDATIONS. A number of theories has been developed for estimating the maximum load that ground is capable of supporting, notable those of Prandtl and Terzaghi. These are described in Chapter 22.

Construction Problems

1·20 Where roads have to be constructed over foundations with insufficient bearing capacity, the following alternative methods of treatment may be adopted:—

(1) The complete excavation of the soft material if its depth is not greater than about 5 ft.

(2) The displacement of the soft material by blasting and replacement by granular material (bog-blasting).

(3) The spreading of the load by the use of fascines, etc.

The construction of embankments on unstable foundations is dealt with in Chapter 25.

THE CUTTING

Structural Problems

1·21 STABILITY OF SLOPES. The main structural consideration with a cutting is the stability of its slopes. The possibility of the occurrence of slips is greater with cuttings than with embankments, since it is common with cuttings for water to seep towards the slopes under hydraulic gradients. Drainage measures are thus frequently required. In the case of cuttings in silty sands electrical drainage has been employed to stabilize the slopes during excavation. In such cases the effect of the electric current has been to reverse the direction of the hydraulic gradient of the seepage flow. This subject is described in more detail in Chapter 17.

1·22 RISE IN LEVEL OF FLOOR OF CUTTING. Clays and other highly compressible soils are known to swell when an overburden pressure is removed, the effect being the reverse of the consolidation process. Roads constructed in clay cuttings where a considerable weight of overburden has been removed are liable to rise above their original level. In a recent investigation by the Road Research Laboratory a rise in level of almost 4 in. was noted in the case of a concrete road in a clay cutting. Deleterious differential movement of a road surface is particularly likely to occur at the junction of a clay cutting and fill, especially if the latter is poorly compacted.

1·23 The laboratory consolidation test may be used to estimate the rise liable to occur to the floor of a clay cutting due to the removal of a known weight of overburden.

Construction Problems

1·24 The problems arising in the construction of cuttings are mainly concerned with the excavation of the soil. Difficulties in excavation may be anticipated by making a detailed soil survey of the site of the cutting. A soil survey helps in three ways:—

(1) The soil types to be encountered during the excavation can be identified, thus enabling the most suitable means of excavation to be selected.
(2) A knowledge of the soil types enables the correct slope of the cutting to be determined. Cases are known where the unexpected occurrence of an unsuspected type of soil has necessitated a considerable modification to the original design of the cutting.
(3) The level of the water-table can be determined, thereby assisting in the planning of the necessary drainage measures.

The method of making a soil survey is described in Chapter 8.

THE ROAD (AIRFIELD) PAVEMENT

1·25 The pavement is the hard crust placed on the soil formation after the completion of the earthworks. Its main functions are:—

(1) To provide a smooth riding surface.
(2) To distribute the traffic loads over the soil formation sufficiently to prevent the soil from being over-stressed.
(3) To protect the soil formation from the adverse effects of weather.

1·26 The characteristics of the pavement are thus dependent not only on the nature of the traffic but also on the properties of the soil on which the pavement is constructed. The soil foundation which directly receives the traffic loads from the pavement is known as the subgrade.

1·27 The main structural element of the pavement is the base and its function is to distribute the traffic loads. This is often placed directly on the subgrade, but sometimes a thin layer of material known as the sub-base intervenes. A sub-base is usually employed with hand-pitching and similar bases where there is a possibility of the subgrade working up into the base.

1·28 The base may be surfaced with either a concrete or a bituminous surfacing, in which case the pavement is sometimes described respectively as " rigid " or " flexible." These terms apply particularly to the subject of pavement design, since two distinct approaches are used in determining the required thickness of pavement, depending on whether a concrete course is included or not.

1·29 There are many methods of pavement design. They are all more or less empirical, and usually consist of a relationship, in the form of either a graph or a formula, between the required thickness of construction, the wheel load of the traffic and a measured value of some property of the subgrade soil. A variety of tests is used to measure the soil property, including shear strength tests, penetration tests and soil classification tests. Methods of pavement design are described in Chapter 20 and associated methods of measuring the strength of soil are dealt with in Chapter 19.

THE SUBGRADE

Structural Problems

1·30 A satisfactory subgrade is able to resist the effects of both traffic and weather. A reduction in the supporting power of the subgrade due to either of these causes is sometimes referred to as regression.

1·31 EFFECT OF TRAFFIC. The principal way in which traffic causes a loss of subgrade support is thought to be by the compaction of the soil. This may take the form of a local reduction in volume of the soil which results in the differential settlement of the subgrade. An example of this effect is the settlement of roads over poorly reinstated trenches. Sandy subgrades are particularly susceptible to compaction, especially on airfields where the vibration from aircraft has a compacting effect on the sand. Clay subgrades however may also be liable to plastic deformation under repeated loading.

1·32 A loss in subgrade support causes bituminous roads to develop crazing and unevenness in the surfacing and concrete roads to become severely cracked. In Chapter 21 a description is given of a method of investigating the causes of such defects.

1·33 EFFECT OF WEATHER. As the subgrade is close to the surface, it usually lies within the zone which is affected by weather. The two principal effects are frost action in the subgrade and seasonal weather changes leading to fluctuations in the moisture content of the subgrade.

1·34 FROST ACTION IN THE SUBGRADE. If frost penetrates to the subgrade, the road surface may be lifted irregularly by several inches. At the same time sufficient moisture may be drawn to the frozen zone to cause a considerable softening of the subgrade when the thaw takes place; this softening may result in the complete disintegration of the road surface. Silty sands and soft chalk are the chief foundations affected by frost. This subject is dealt with in more detail in Chapter 18.

1·35 EFFECT OF SEASONAL WEATHER CHANGES ON THE MOISTURE CONTENT OF THE SUBGRADE. In winter the soil verges of a road are usually wetter than the subgrade while in summer they are usually drier. The effect of these changes in moisture content in the verges is often found to extend for some distance under the edges of the road into the subgrade. When the subgrade is composed of a heavy clay these seasonal fluctuations in moisture content are accompanied by corresponding changes in the volume of the soil. In such cases the edge of the road has been found to be subject to a seasonal rise and fall, with respect to the centre of the road, of as much as 2 in. During periods of drought these movements are greater and may result in the severe cracking of bituminous surfacings and loss of shape of concrete roads. Further information on this subject is given in Chapter 16.

Construction Problems

1·36 It is clear from the foregoing that in the preparation of the subgrade the aim should be to provide adequate resistance to the action of traffic and weather. The main requirements are:—

(1) To obtain adequate compaction of the soil.

(2) To maintain the subgrade in a stable condition at a constant moisture content.

(3) To protect the subgrade from frost.

1·37 SUBGRADE COMPACTION. Soil compaction is fully dealt with in Chapter 9. Adequate compaction reduces the extent of subsequent deformation of the subgrade and the rate of water absorption by the soil. The only case where it is unlikely to be advantageous is for clay in cuttings, where the destruction of the natural structure of the undisturbed soil results in a loss of strength.

1·38 SUBGRADE DRAINAGE. The main object of subgrade drainage is to exclude water from the subgrade and not to remove moisture from the soil: a reduction in the moisture content of the subgrade may cause as much damage to the road surface as an increase in the moisture content. The ideal situation is to have the subgrade in a satisfactorily stable condition when the road is constructed and subsequently for the subgrade to remain at a constant moisture content. Subgrade drainage is discussed in Chapter 17.

1·39 Assuming efficient subgrade drainage and ignoring the effect of the seasonal wetting and drying of the verges, the subgrade will tend to reach an equilibrium moisture content which is dependent on the level of the water-table and the overburden pressure imposed by the road pavement. It is particularly important to be able to estimate this equilibrium moisture content in any particular set of circumstances since the design thickness of the road

should be based on measurements of soil strength made at this moisture content. The present state of knowledge of the factors controlling the movement of moisture above the water-table is outlined in Chapter 16.

1·40 PREVENTION OF FROST DAMAGE. In this country frost rarely penetrates to a depth greater than 1 ft. Little trouble due to frost damage in the subgrade need be experienced if a thickness of construction of 1 ft or more is employed on frost-susceptible soils.

THE BASE

Structural Problems

1·41 As already mentioned, the function of the base is to spread the traffic loads sufficiently to prevent over-stressing of the subgrade. The main considerations are:—

(1) Thickness.

(2) Stability under traffic loads.

(3) Resistance to weathering.

1·42 Reference has previously been made to the necessity of relating the thickness of road crust to the traffic and subgrade conditions. The thickness of base is estimated in a similar way, but allowance is made for the thickness of the surfacing.

1·43 To be effective, the base should possess considerably greater resistance to deformation than the subgrade. Cases have been reported of road failures due to the inadequate compaction of the base. Care is particularly necessary, therefore, when materials that are hard to compact, such as hardcore, are employed.

1·44 The base should also possess considerable resistance to weathering, since it has less protection than the subgrade. Materials whose stability is affected by water should be avoided, where possible, and frost-susceptible materials should be excluded entirely.

Construction Problems

1·45 The conventional forms of base are hand-pitching, hardcore and macadam. These are suitable in areas where there is a plentiful supply of rock and gravel. Where this is not the case, methods of base construction with soils and low-grade aggregates are now being increasingly employed. These processes are described as " soil stabilization." They are finding considerable application not only in the undeveloped areas of the colonies but also in this country where they have been used for constructing housing estate roads and the bases beneath concrete surfacings for main roads. Soils may be stabilized:—

(1) Mechanically (Chapter 11).

(2) With cement (Chapter 12).

(3) With bituminous materials (Chapter 13).

(4) With resinous and other materials which waterproof the soil (Chapter 14).

The methods of construction used in soil stabilization are described in Chapter 15.

SOIL—ITS CLASSIFICATION AND COMPOSITION

1·46 In view of the wide diversity of soil types, it is desirable to be able to classify them into groups possessing similar physical properties and many methods have been developed for this purpose. The more important of these are reviewed in Chapter 4. Soils are usually classified on the basis of simple laboratory tests such as the particle-size analysis and the plasticity tests. Descriptions of these tests are included in Chapter 3.

1·47 It has been thought desirable to include in the book a simplified picture of the nature of soil and of the characteristics of its various components; this is given in Chapter 2. Methods of measuring the proportions of some of the more important chemical constituents in soil are described in Chapter 5.

Rock

1·48 Rock rarely presents any problem as a foundation for roads, except in the case of soft varieties such as chalk and limestone. In view of the wide occurrence of chalk in southern England, a whole chapter (Chapter 7) has been devoted to a discussion of its properties of interest to road engineers.

1·49 Rock is mainly of interest on account of its wide use in foundations (in the form of hand-pitching, hardcore, etc.) and as the main structural element of concrete and bituminous surfacings. The principal facts of the geology, production and testing of roadmaking aggregates (crushed rock and gravel) are given in Chapter 6.

CHAPTER 2

THE NATURE OF SOILS

INTRODUCTION

Phases of the Soil

2·1 Soil is an assemblage of solid particles forming a porous structure: depending on the circumstances these pores may contain water or air or both. The composition of a soil in terms of these phases can be conveniently represented by a point on the triangular chart shown in Fig. 2·1, in which the ordinates give the volume percentages of the three constituents. Any alteration of the composition of the soil can also be followed on the chart; line A represents the phase changes during a soil compaction test, line B shows the change in state of a soil sample during a volumetric shrinkage test, and line C similar data for a consolidation test.

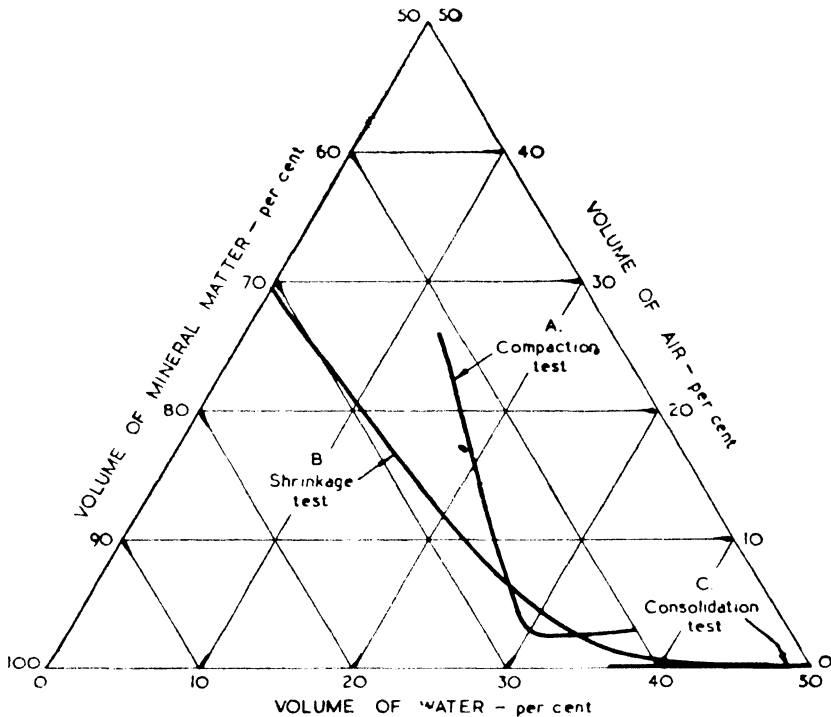



FIG. 2·1 TRIANGULAR CHART FOR REPRESENTATION OF PHASE COMPOSITION OF SOIL

2·2 Although the chart in Fig. 2·1 indicates the composition of the soil *in terms of the volumes of the constituents, in practice the amount of the* solid component present is usually expressed as the weight in unit volume of

C

the bulk soil, i.e. the number of pounds per cubic foot or grams per cubic centimetre, since it is more convenient to measure the weight of solid material than the volume in a given space. The weight of soil solids contained in a unit volume of the bulk soil is commonly referred to as the dry density and this may be misconceived as being obtained by determining the volume of a given weight of soil after drying. It is, in fact, intended to signify the weight of solid matter present in a unit volume of soil after the hypothetical removal of the water without bulk volume change.

2·3 The moisture content of the soil is expressed as a percentage of the weight of the solid material in the soil, i.e.

$$\text{Moisture content} = \frac{100\text{m}}{\text{w}} \text{ (per cent)}$$

where m is the weight of water and w the weight of the solids in the same units.

2·4 It is impracticable to express the air content of the soil on a weight basis, so this constituent is usually given as a percentage of the total volume of soil, i.e.

$$\text{Air content} = \frac{100\ V_a}{V_s + V_m + V_a} \text{ (per cent)}$$

where V_s, V_m and V_a are the volumes of solid material, water and air respectively in similar units.

2·5 The volumes occupied by air and water (liquid) in a soil are often referred to as the voids, and the sum of the two volumes expressed as a percentage of the total volume of the soil is known as the porosity, i.e.

$$\text{Porosity} = \frac{100\ (V_a + V_m)}{V_s + V_m + V_a} \text{ (per cent)}$$

while the ratio of the sum of these volumes to the volume of the solid component is called the voids ratio (e), i.e.

$$\text{Voids ratio} = e = \frac{V_m + V_a}{V_s}$$

THE AIR IN SOIL

2·6 Although the air in soil is important from the agricultural point of view, since it is necessary to support vegetation, for engineering purposes the volume of the air should be reduced as much as possible, since it contributes nothing to the strength of the soil as a whole.

Aerobic Bacteria and Fungi

2·7 Many types of micro-organisms are present in the soil, living on the organic materials which accumulate in the surface layers. A large proportion of these, the so-called aerobic bacteria and fungi, require access to the oxygen and nitrogen of the soil air as a condition of their existence. In civil engineering

they are mainly of interest because they can attack and destroy organic compounds in constructional materials in contact with the soil. The jute hessian, for instance, employed in prefabricated bituminous surfacings (P.B.S.) rots because of micro-biological attack, and certain resins, e.g. " Vinsol," which are added to soil to waterproof it, are also attacked. The latter phenomenon has been investigated for the Road Research Laboratory by Jones[1], who believes that anaerobic conditions, i.e. the exclusion of air from the soil, reduce the attack.

Moisture Movement in the Vapour Phase

2·8 At moisture contents well below saturation, the air-spaces inside the soil can provide continuous passages through which water may move in the form of vapour, and consequently it is possible for changes in the moisture content of soil to occur owing to the movement of water vapour from one region of the soil to another through these air channels. This movement is due to differences in the relative humidity of the water vapour in different parts of the soil. (The relative humidity of the water vapour is defined as the pressure of the water vapour in soil expressed as a percentage of the saturated vapour pressure of water at the same temperature).

2·9 The factors on which the relative humidity of water vapour in soil depends are discussed in detail in Chapter 16. Briefly, however, differences in relative humidity are associated with variations in soil type, soil moisture content and temperature. The temperature is the only one of these factors likely to be of importance under practical road conditions in this country, since local variations in moisture content can only cause appreciable differences in relative humidity when the soil has a comparatively low moisture content (below about 4 per cent for sands, and below about 10 per cent for clays). Temperature gradients such as those created in the soil by the daily and annual temperature cycles may cause considerable differences in vapour pressure within the top few feet of the soil, and if free channels are available in which the water vapour can move, a transfer of moisture will occur.

2·10 Under the climatic conditions of the British Isles, however, when the soil is normally close to saturation, free movement of water vapour is largely prevented and appreciable transfer of moisture is unlikely to occur. On the other hand, vapour movements may be of considerable importance in tropical and arid areas where very low moisture contents and large temperature variations are experienced in soil. This is probably the explanation of the high moisture contents which are found under some roads constructed in arid areas. An impervious surfacing in such roads is able to prevent the evaporation by which the moisture accumulated in the surface layers of the soil is normally removed.

THE WATER IN SOIL

2·11 Water plays an important part in determining the physical properties of a soil, and most of the classical studies in soil mechanics, such as those on consolidation, stability and compaction, are concerned in some way or other with the liquid/solid relationships of soil. Water also acts as a solvent for any salts in the soil.

Effects due to the Liquid Characteristics of Water

2·12 COHESION. Fine-grained soils, such as silts and clays, owe the greater part of their mechanical cohesion to the fact that individual particles in the soil mass are bound together by films of water. The cohesive forces that arise from these films are thought to be of two types, viz. those due to surface tension forces at the air/water interfaces within the soil structure, and those due to interaction between the soil particles or between the particles and water molecules.

2·13 The cohesion due to the surface tension of water arises in soils at low moisture contents, as the soil then contains the necessary proportion of air-voids. The theoretical conception of the cohesion in this condition has been postulated by Haines[2], and is illustrated in Fig. 2·2 in which two "ideal" spherical particles of the same radius are joined by an annular film of water. The surface tension acts in directions tangential to the surfaces of the particles, drawing them together and causing a pressure deficiency in the water film which also draws the particles together. The total force of attraction between the two particles is given by the equation:—

$$f = \frac{2 \pi a T}{1 + \tan \frac{\theta}{2}}$$

where T = surface tension,
a = radius of particle,
θ = angle as indicated in Fig. 2·2.

It follows that the cohesion should increase as θ decreases, that is, as the water film decreases in magnitude. This indicates the reason for the increase in cohesion due to surface tension forces as the soil is dried.

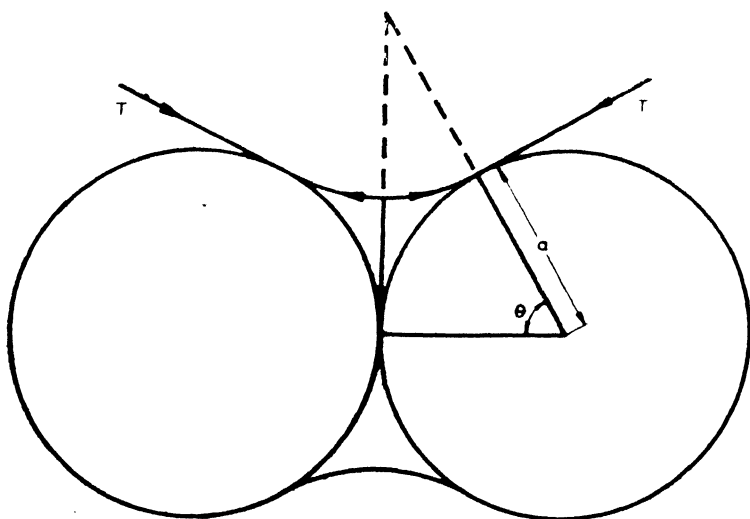



FIG. 2·2 COHESION BETWEEN SPHERICAL PARTICLES (Haines)

2·14 If the cohesive stress per unit area is calculated from the equation above, it is found to be proportional to T/a, i.e. it increases with increasing surface tension and with decreasing particle size. Consequently, as has been found by experience, cohesion should be most marked in soils, such as clays, which contain a high proportion of very fine particles.

2·15 Although Haines' theory applies to ideal spherical soil particles, it is now known that many of the finer particles in the clay fraction are flat and plate-shaped, and Nichols[3] has developed a formula for the cohesive force for this case, i.e.

$$f = \frac{4c\pi r T}{d}$$

where c is a constant and d the distance apart of the two plate-shaped particles. It is found that such particles have a greater mutual cohesion than spherical particles, owing to the greater area of film that can be developed between them. A theoretical conception of cohesion involving electrically charged ions in the soil water has been developed by Russell[4] in which the soil water acts as a bonding agent. In the water there are positively charged ions (cations) such as Na^+, Ca^{++} and Al^{+++} sufficient in number to balance the negative charges on the soil particles, thus making the system electrically neutral. These cations are also hydrated to various degrees, and give rise to chains of oriented water molecules. Where such a hydrated cation is in the vicinity of a soil particle the two sets of water molecule chains link up, providing a bond between the ion and the particle surface. Cross-linkage may also occur, whereby an ion between two adjacent soil particles may act as a bridge between them (Fig. 2·3).

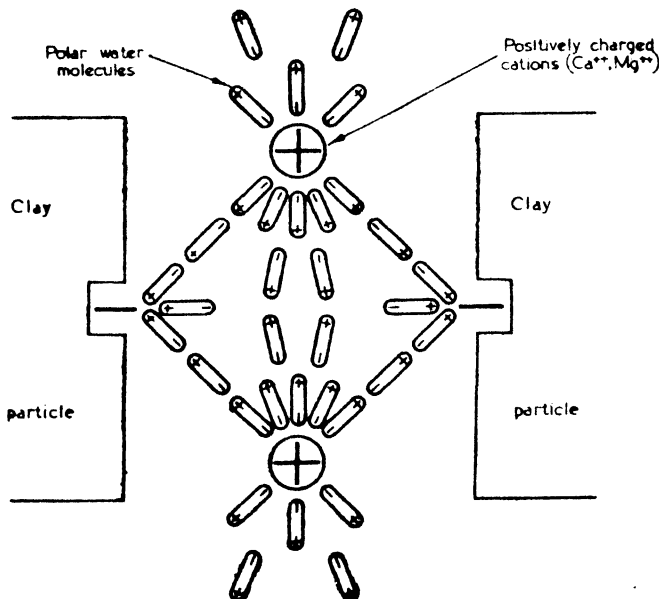



FIG. 2·3 COHESION DUE TO PARTICLE HYDRATION (Russell)

2·16 It will be seen that the type of cohesion envisaged by Russell is essentially a surface effect, and will therefore be particularly marked in clay soils, which have a considerable surface area per unit weight. It also depends on the type of ions present in the water, and on the electrical characteristics of the particle surfaces, i.e. on the chemical composition and structure of the particles.

2·17 The cohesive forces so far discussed are believed to operate in soils at relatively low moisture contents. It is known that the cohesion of soil decreases rapidly as the moisture content increases, but the nature of the forces between the particles at high moisture contents is still a matter of speculation. It is believed however, that they are functions of Van der Waal forces of attraction between the particles, and electrostatic forces of repulsion due to the charges associated with the particles.

2·18 SUCTION. It has been noted in the previous section that water molecules can be associated with the surfaces of soil particles, and it is generally considered that the former are in an adsorbed state, i.e. the particle surfaces are hydrated. The forces causing this hydration, together with the surface tension at the air/water interfaces previously referred to, combine to produce a state of reduced pressure or suction in the water, the value of which is dependent on the moisture content of the soil. The suction/moisture content relationship is found experimentally to be continuous for all soils, the suction increasing rapidly with decreasing moisture content.

2·19 In clays which are normally saturated at moisture contents above about 15 per cent it follows from the preceding discussion that the suction in the water is due mainly to particle hydration, whereas in granular soils surface tension plays the more important role. The suction/moisture content relationship for soils is dealt with in greater detail in Chapter 16.

2·20 SWELLING. An effect associated with particle hydration is the swelling of clay soils. At relatively short distances from the surfaces of clay particles the orienting and adsorbing forces acting on the water molecules are very strong, and the water is believed to be in the solid rather than in the liquid state (adsorbed water)[5] As these adsorbed layers grow during the wetting of a clay, the effective solid volume associated with each particle increases, and if the layers are in contact with each other, the growth of the individual layers will be reflected in an increase in the total volume of the soil structure.

2·21 In practice, the adsorbed water films in clay grow in thickness until the suction pressure in the water becomes equal to the overburden pressure on the soil due either to self-loading or to externally applied loads. If, when this equilibrium is reached, the loading is increased, the adsorbed water films are reduced in thickness and settlement occurs. This process is termed consolidation and is dealt with in Chapter 23. Structures built on clay soils liable to moisture changes normally rise and fall with the changing moisture conditions.

2·22 SHRINKAGE. Shrinkage in clays may result from external loading (consolidation) but it is often associated with the loss of moisture due to evaporation or transpiration from vegetation. A typical curve relating the volume of an initially saturated soil block to its moisture content during shrinkage is shown in Fig. 3·7. It will be seen that the curve consists of two parts, the first where the volume of the soil is a linear function of the amount

of moisture lost, and the second where the decrease in volume is less for a given loss of moisture, and the relationship is non-linear. During the first part of the experiment the decrease in volume of the soil is equivalent to the volume of moisture lost, and the system remains saturated, but later air begins to enter the soil and the total volume change is relatively smaller. If the sloping straight line is extrapolated to cut the abscissa of volume at zero moisture content, the moisture content corresponding to the point of intersection is known as the shrinkage limit (SL), i.e. the moisture content below which little shrinkage should take place. For most British soils this value is fairly constant at about 12 to 14 per cent, equivalent to a voids content of about 30 per cent. This voids content corresponds approximately to a theoretical porosity of 26 per cent which has been calculated to occur in a closely packed structure consisting of single-sized spherical particles[(6)]. A constant shrinkage limit would therefore be expected for all soils if the shape of the particles were similar and independent of particle size.

2·23 PLASTICITY. When a mass of soil is subjected to a stress above its elastic limit, it will be deformed and ruptured. If the soil is cohesive, however, and if its moisture content is high enough, deformation is not accompanied by a breaking up of the structure but plastic flow takes place instead. Plasticity is a characteristic of all cohesive soils, and the relationship between the plastic properties of a soil and its constitution and mechanical performance are of considerable importance in soil classification.

2·24 Plasticity in soil is due to the lubricating effect of the water films between adjacent particles. Thus it is dependent on the factors which influence the area and thickness of these films, i.e. on the size and shape of the individual particles and the chemical nature of their surfaces. However, the thickness of the films is primarily dependent upon the moisture content of the soil, and the plasticity characteristics of soils are therefore generally studied by determining the amounts of moisture required to bring them to arbitrarily defined states of plasticity. The methods by which this is done vary in the different branches of soil technology but the methods originally devised by Atterberg[(7)] for agricultural use have won wide acceptance in soil engineering.

2·25 These are based on the supposed existence of two transition points in the state of the soil as the moisture content is increased from dryness. The first, the plastic limit, is the moisture content at which the soil passes from the solid to the plastic state as defined by a given procedure, while the second, the liquid limit, indicates the moisture content at which it passes from the plastic to the liquid state, also arbitrarily defined. The difference between the two limits gives the range of moisture contents over which the soil is plastic, and is referred to as the plasticity index. The experimental details of the two tests are given in Chapter 3.

2·26 Both the liquid and plastic limits are dependent on the clay fraction of the soil, as may be seen from Fig. 2·4 which shows the results of a large number of tests on a group of clay soils, plotted against the clay contents. A soil with a high clay content usually has high liquid and plastic limits, while a less cohesive sandy soil gives low test results. Liquid limits below 20 per cent are exceptional and difficult to determine experimentally, while most clay soils have liquid limits of the order of 50 to 90 per cent. When a soil contains

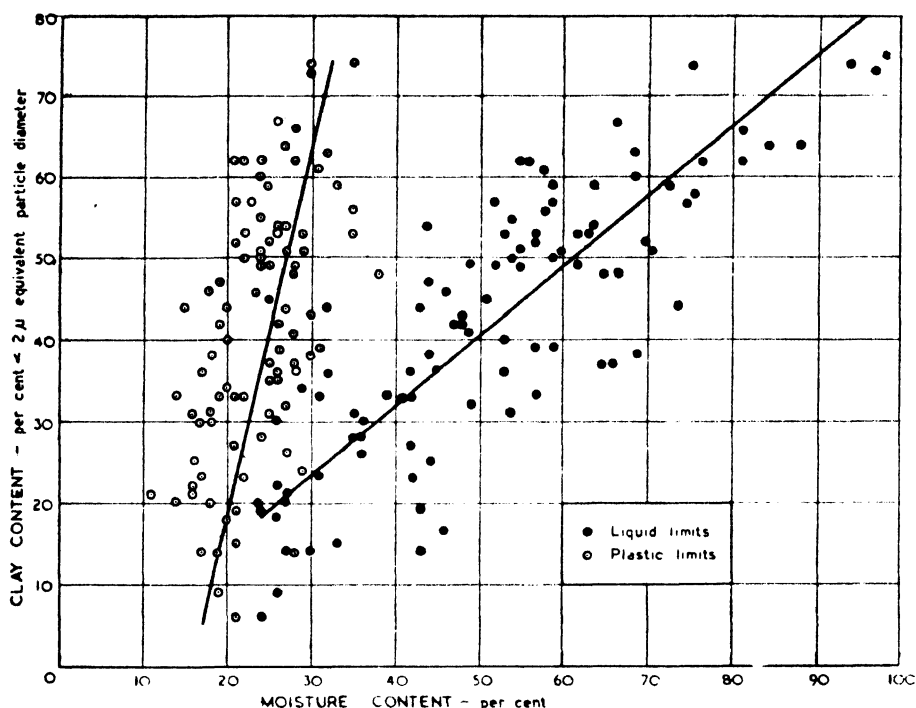



FIG. 2·4 RELATIONSHIP BETWEEN CLAY CONTENT AND LIQUID AND PLASTIC LIMITS FOR A GROUP OF BRITISH CLAY SOILS

a high proportion of organic matter, both the limits are higher, although the plasticity index is similar to that of the same soil without an organic admixture.

2·27 Generally, it may be said that the plasticity index is a function of the amount of clay present in a soil, while the liquid and plastic limits individually are functions of both the amount and type of clay. Consequently, if liquid limits are considered in relation to plasticity indices, any differences in the relationships between them will be due to differences in the type of clay. Exceptions to this are soils containing much organic matter, and those whose particles are porous and contain voids, both of which have relatively high liquid limits for a given plasticity index.

2·28 This relationship between liquid limit and plasticity index has been used by Casagrande[8] as the basis of a classification system for cohesive soils. A modified form of this chart is shown in Fig. 4·2. It consists of a graph relating plasticity index and liquid limit, on which the main dividing axis is a line corresponding to the equation:—

$$PI = 0{\cdot}73\,(LL-20)$$

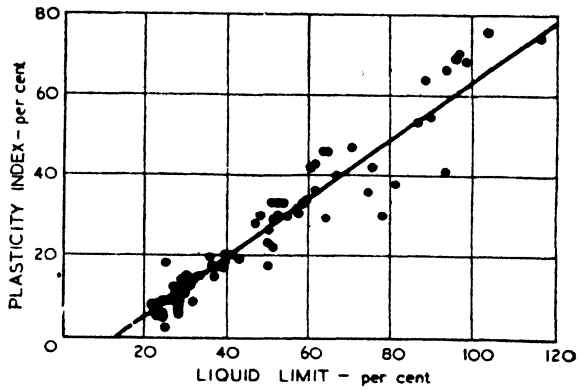
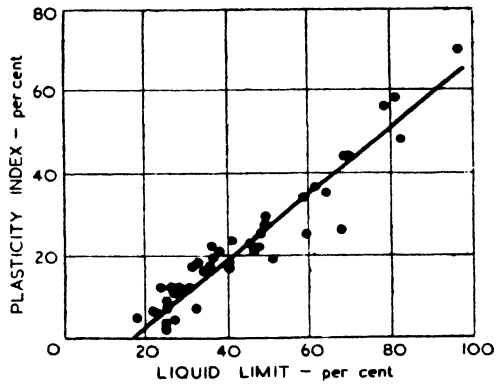
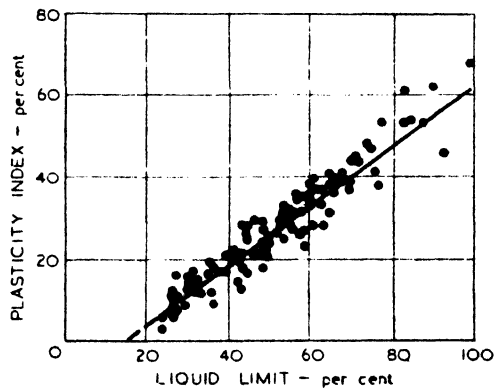
where PI = plasticity index and LL = liquid limit.

(a) Oligocene and Eocene

(b) Upper and Lower Greensand and Gault

(c) Oolitic and Liassic

FIG. 2·5 RELATIONSHIPS BETWEEN PLASTICITY INDICES AND LIQUID LIMITS FOR THREE GROUPS OF BRITISH SOILS

2·29 The results of tests on inorganic soils usually give points which lie above and to the left of this line, while soils containing much organic matter tend to give points lying below and to the right of the line. Ordinates at LL = 35 and LL = 50 further subdivide the chart. Code letters which have been assigned to different areas can be used to identify soils from the test results, and to predict their probable behaviour in the field from data previously acquired with similar soils.

2·30 In Fig. 2·5 the plasticity indices and liquid limits of three groups of British soils are plotted in the same way, showing that lines similar to Casagrande's A-line are obtained. The equations for a number of groups of relatively organic-free soils from the British Isles have been determined at the Road Research Laboratory, and have been found to vary between approximately PI = 0·72 (LL—15) and PI = 0·82 (LL—17). Thus it appears that most British soils of this type fall into the CH, CI or CL groups. A more detailed discussion of the plasticity tests in relation to soil classification is given in Chapter 4.

2·31 COMPACTION. Another effect that is dependent upon lubrication of the soil particles by moisture is that of compaction, in which the particles are made to pack more closely together through a reduction in the air-voids, generally by mechanical means. Thus, when soil is compacted at increasing moisture contents into a mould by a given amount of work, it is found that the dry density rises to a maximum and then decreases, giving a relationship of the type shown in Fig. 9·1. It is assumed that when the soil is dry, the inter-granular friction prevents the particles sliding over each other and so achieving a minimum volume when subjected to the compacting force. The addition of moisture to the soil, however, lubricates the points of contact and enables the particles to be forced gradually into a denser state. In the limit, however, the voids become filled with moisture and further additions displace the particles, giving rise to a lower dry density.

2·32 PERMEABILITY. In addition to suction, other hydrostatic forces may be developed due to gravity, externally applied pressure and ice formation, which give rise to the movement of moisture in soil. The rate at which this movement occurs is affected partly by the magnitude of these forces, and partly by the resistance offered by the soil to the passage of water through it, i.e. the permeability. The permeability of a soil influences its drainage and consolidation properties, and its susceptibility to frost damage. Its bearing on these phenomena is discussed in Chapters 16, 17, 18 and 23.

Effects due to the Solvent characteristics of Water

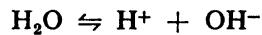
2·33 In addition to its purely physical properties the water in soil possesses characteristics of interest from the engineering point of view, due to its action as a solvent.

2·34 This solvent action is manifested in the actual formation of soil from the parent rocks. Thus, while the latter are initially broken down in size by the primary weathering processes which are mainly physical in nature, the subsequent weathering of the comminuted rock fragments and the translocation of the various elements contained in them is brought about mainly by the chemical action of water during leaching. However, it is sufficient to

note here that leaching brings certain materials into solution. These fall roughly into two groups, viz. (a) soluble salts, (b) organic matter.

2·35 The soluble salts are dissociated in solution giving rise to positively charged ions (cations) of such metals as sodium, magnesium, calcium and aluminium, which have the property of being adsorbed on to the surfaces of the soil particles. Such ions are often referred to as exchangeable bases, and the nature of the exchangeable bases in a soil can influence its physical properties to a considerable extent. A fuller discussion of this phenomenon is given later.

2·36 The hydrogen ion (H^+) also occurs in all solutions, and is usually associated with a corresponding amount of the hydroxyl ion (OH^-), since both result from the dissociation of water according to the equation:—

$$H_2O \leftrightharpoons H^+ + OH^-$$

2·37 In completely neutral water, the hydrogen and hydroxyl ions are present in equivalent concentrations and the solution is said to have a " neutral " reaction. However, when the concentration of hydrogen ions present is relatively larger than that of hydroxyl ions the solution is acid.

2·38 In order to express quantitatively the acidity or alkalinity of a solution, use is made of the pH scale; here the pH value is given as the logarithm to the base 10 of the reciprocal of the hydrogen ion concentration. On this scale neutral solutions have pH values of 7·0 while acids have lower values and alkaline solutions have higher values numerically. The pH scale can be used in assessing the reaction of water in soil and suitable methods of measurement have been devised for this purpose. These include the measurement of the electric potential developed between suitable electrodes immersed in a soil suspension of given concentration (soil/water ratio = 1:3), and the comparison with known standards of the colours developed by certain dye-stuffs when added to a clear soil suspension. A detailed description of a method of the latter type is given in Chapter 5.

2·39 In addition to giving rise to metallic ions, the soluble salts themselves may affect in various ways the soil or engineering structures in contact with it. These ways are as follows:—

(1) By attacking concrete and other materials containing cement.

(2) By disrupting porous materials such as the soil itself by crystallization.

(3) By corroding metals, such as iron pipes.

2·40 The salts concerned are usually the sulphates, principally those of sodium, magnesium and calcium. Calcium sulphate occurs naturally in soils, usually clays, as crystalline gypsum. Sodium and magnesium sulphates occur to a lesser extent in the British Isles, but as they are more soluble than the calcium salt they are potentially more dangerous.

2·41 The attack of materials containing cement is believed to be due to the formation of calcium sulpho-aluminate[9], as a result of a reaction between the sulphate and the aluminate compounds in the cement. This compound is

highly hydrated and contains 31 molecules of water of hydration. The internal stresses in the material containing the cement, created by the expansion accompanying the formation of calcium sulpho-aluminate, are sufficient to cause disruption of the cement matrix, and mechanical failure of the material as a whole.

2·42 The simple crystallization of some soluble salts is also known to have a damaging effect on porous materials. This is of importance in arid areas, when considerable upward movement of moisture can occur in soils. Dissolved salts such as sodium sulphate may be transported with this moisture and concentrated in the surface strata of the soil, where the salts crystallize and disrupt the soil structure, creating what are known as " salt boils."

2·43 An example of the detrimental effects of salt crystallization can be seen in Plate 2·1A which shows a cylindrical specimen of soil-cement mixture after standing for some months in a shallow depth of water. Moisture has moved up from the base of the specimen and evaporated at the top surface, causing an accumulation of soluble salts, of which the soil contained a high proportion. The crystallization of these salts had disrupted the top surface, reducing it to a powder.

2·44 Soluble sulphates are also a major factor in the virulent type of corrosion of metal pipes occurring in waterlogged clay soils. There is strong evidence that this type of corrosion is due to the activities of anaerobic sulphate-reducing bacteria [10] (generic name: *desulphovibrio desulphuricans*). It is believed that these organisms are able to reduce the sulphates in soil by using the hydrogen discharged at the cathodic elements of galvanic cells set up on the surface of metals; polarization is thus prevented and the corrosion process can proceed in the virtual absence of oxygen. Of a large number of cases of corrosion of buried pipes investigated by the Chemical Research Laboratory (D.S.I.R.), the majority were found to have occurred in clay soils and to be associated with microbial activity requiring the presence of sulphates.

2·45 Organic matter can also be dissolved by water passing through soil, although it is not known whether it is present as a true solution or in the colloidal form. Thus organic matter may be leached out of the topsoil and deposited in the subsoil below, resulting in dark zones. It is possible that soluble organic matter also influences the re-distribution of mineral elements of the soil, since iron is known to form soluble complexes with certain organic compounds. Thus iron may be removed from some parts of the soil and deposited lower down in the form of concretions round siliceous particles, giving rise to an " iron-pan."

THE SOLID MATERIAL IN SOIL

2·46 The solid phase in soil is composed of a mixture of matter derived from the physical and chemical weathering of rocks, and organic matter which consists of the more or less decomposed remains of plant and animal organisms. These two groups are so different in their origins and properties that it is convenient to consider them separately in detail.

Organic matter

2·47 The organic matter in soil is derived from either plant or animal remains which are added to the soil when the organisms die, and which subsequently undergo decomposition at different rates due to chemical and bacterial action. The fraction of animal origin is relatively small and does not tend to accumulate in the soil, since it is decomposed fairly rapidly and the decomposition products utilized as nutrient compounds by living plants. The fraction of vegetable origin, on the other hand, is relatively large, and persists in the soil for a longer period owing to the comparatively higher resistance of some plant materials to decomposition. The total amounts of both kinds of organic matter in a soil are a function of the rate of supply from dead organisms and the rate of decomposition into products which are subsequently removed from the soil.

2·48 Since it is derived from organisms living on or near the surface of the earth, organic matter tends to be concentrated in the top 2 to 12 inches of the soil, under normal conditions. Leaching in sandy soils may, however, cause soluble constituents to be extracted and deposited lower down while the activity of earthworms may also extend the depths to which organic admixture can occur. The distribution of organic deposits such as those of peat, lignite or coal is conditioned by geological factors, and may extend to much greater depths.

2·49 The composition of the organic matter is dependent on the plant cover and on the extent to which decomposition has progressed. Thus, the organic constituents of forest soils are derived largely from leaf-mould, whereas in pasture soils the organic matter is derived mainly from grass leaves and roots. In some cases the organic matter may contain plant constituents that can be recognized visually, while in other cases the decomposition may have proceeded so far that the original plant structure has completely disappeared, leaving a dark amorphous material called " humus." Freshly decomposed organic matter and humus have somewhat different characteristics in that the former consists of macro-particles or fibres that are relatively inert from a physical or chemical point of view, whereas humus is acidic and colloidal in nature, has a high base exchange capacity and absorbs water readily with a very considerable increase in total volume. It is believed to be a complex compound formed from lignins and proteins from plants, the detailed composition of which may vary from soil to soil.

2·50 From the engineering point of view, organic matter has undesirable characteristics, the chief of which are the open, spongy structure and the mechanical weakness of the constituents. It undergoes considerable volume changes when subjected to load or changes in moisture content. The natural moisture content may also be very high (100 to 500 per cent) so that the mechanical stability may be very low. The acidic nature of the constituents tends to give an acid reaction to the water in soil which in turn may have a corrosive effect on materials buried in the soil.

2·51 Soils containing appreciable amounts of organic matter are generally removed from a site prior to construction, if this is practicable. When this cannot be done, as in the case of deep peat deposits, and it is impossible to re-locate the road, satisfactory construction has been achieved for light traffic

either by using fascines or by making the road structure as light as possible and allowing it to " float " on the peat.

2·52 It is not known at what concentration the organic matter begins to affect the characteristics of a soil. Significant effects, probably of a chemical nature, have been observed in connexion with cement stabilization when as little as 0·5 per cent by weight of organic matter has been present, but the physical characteristics of a soil are not normally influenced until the concentration rises above 2 to 4 per cent.

2·53 Several methods are available for the determination of organic matter in soil, based either on the loss in weight of the soil when the organic matter is destroyed, or on a determination of the percentage of organic carbon present, which is assumed to constitute a relatively constant proportion (about 58 per cent) of the organic matter. These methods are described in detail in Chapter 5.

Inorganic Matter

2·54 The inorganic or mineral components usually form the main bulk of a soil. They originate from the various types of rocks occurring in the earth's crust, by the soil-forming or " pedogenic " processes, which can be both of a physical and of a chemical nature. The physical or primary weathering processes include disruption of the rock due to differential expansion and contraction following temperature changes, glaciation and abrasion of the rock by wind- or water-borne particles. The secondary weathering processes are of a chemical nature and include the leaching action of water containing dissolved carbon dioxide, which causes the translocation of various chemical constituents in different zones of the soil. The nature of the products of physical and chemical weathering processes is influenced by a number of factors, such as the parent rock, climate, topography, associated vegetation and geological time. For a more detailed discussion of the soil-formation processes and their effects, the reader is referred to specialized works on the subject, e.g. those by Jenny[(11)] and Robinson[(12)].

2·55 The mineral matter in soil usually occurs in the form of solid particles of different types, and the physical characteristics of a predominantly inorganic soil are a reflexion of the properties of its component particles. The more important of these properties are size, shape and mineralogical constitution.

2·56 Size and shape are to some extent functions of the mineralogical constitution, as for example in micaceous soils, where the particles have the laminar structure of the mineral in bulk. A very hard mineral such as quartz gives rise to particles that are less rounded than those derived from softer minerals under similar weathering conditions. The alumino-silicate clay minerals kaolin and montmorillonite occur only in the finer range of particle sizes, probably owing to their mode of formation.

2·57 The property having most influence on the physical characteristics is that of particle size, and with a soil this is evaluated by determining the distribution of size among the particles. It is impossible to determine the size of every particle, so that in practice determinations are made of the quantities of

particles whose sizes lie between sets of arbitrarily defined size limits. These limits are expressed in terms of " equivalent particle diameters" the assumption being made that the particles are spherical. The size range defined by any two limits is referred to as a " fraction " of the soil, and the various fractions are named to correspond to the type of soil which they resemble, e.g. sand, silt or clay.

2·58 A number of systems of size limits has been proposed by different workers, reflecting the practice in the different branches of soil technology. The system used for engineering purposes in this country is that given below:—

- Gravel fraction—particles between 60 mm. and 2·0 mm. equivalent particle diameter.
- Sand fraction—particles between 2·0 and 0·06 mm. equivalent particle diameter.
- Silt fraction—particles between 0·06 and 0·002 mm. equivalent particle diameter.
- Clay fraction—all particles smaller than 0·002 mm. equivalent particle diameter.

Subdivisions exist, embracing coarse, medium and fine fractions for both sand and silt.

2·59 Each size fraction has specific characteristics which will be exhibited by a soil if it contains a sufficient quantity of particles in that fraction. Some of these characteristics are given below.

2·60 GRAVEL. Gravel consists of particles of coarser material resulting from the disintegration of rocks. In the British Isles, these particles have often been transported in water from the original source, and as a result they are worn down by attrition and have rounded shapes.

2·61 SAND. Sands in the British Isles are usually composed of particles of silica or quartz, but some beach sands contain calcium carbonate in the form of shell particles, and glacial sands contain comminuted rock minerals. The particles are visible to the naked eye and feel gritty when rubbed between the fingers.

2·62 The contribution of the sand fraction to the stability of the soil is due to the mechanical interaction between the particles (internal friction), and soils in which the fraction predominates are therefore referred to as frictional soils. The particles lack any considerable cohesion owing to the relatively small influence of the inter-particle water films or of surface effects, and they contribute little to the moisture suction of the soil. The low surface adsorption further excludes shrinkage or swelling of the particles in this fraction.

2·63 Soils that are predominantly sandy often have an open structure and are therefore permeable and well drained. Consolidation effects are relatively small in sandy soils, which are also unsusceptible to frost damage when they are present in road foundations.

2·64 SILT. Silt particles are physically and chemically rather similar to the particles in the sand fraction, the differences being due mainly to the smaller size. Like sand particles their contribution to soil stability is due mainly to internal friction, but the inter-particle water films do confer a degree of cohesion on the soil.

2·65 Predominantly silty soils are very susceptible to frost heave and this is, perhaps, their chief interest to the road engineer. Owing to the higher permeability of silts, consolidation is much less marked than that experienced with clays. Swelling and shrinkage also occur on a much reduced scale.

2·66 CLAY. The particles in the clay fraction differ from those in the other two fractions, both in their chemical constitution and in their physical properties. Chemically, they consist of hydrated alumino-silicates which are formed during the leaching processes to which the coarser particles of primary rock minerals are subjected. Among the minerals that occur in clay particles are forms of kaolinite, montmorillonite and mica. For more detailed information the reader is referred to a paper by Nagelschmidt[13], which also gives an excellent bibliography on clay minerals.

2·67 Physically, the clay particles differ from those in the coarser fractions in that they are flat and elongated, or lamellar, and thus have a much larger surface area per unit weight than would be the case if they were more spherical or cubical in shape. This plate-like shape can be demonstrated in a number of ways, and can be seen in Plate 2·1B, which is an electron microscope photograph of particles from a sample of Gault clay. The transparent nature of the particles indicates their relative thinness.

2·68 The plate-like shape is a major factor causing the plasticity exhibited by clay particles when mixed with water. They are thought to become oriented with their planes parallel, and the moisture films surrounding them enable the plates to slide over each other easily (Fig. 2·6). A change of particle orientation is thought to be the cause of the differences which are found in the behaviour of undisturbed and remoulded samples of clay.

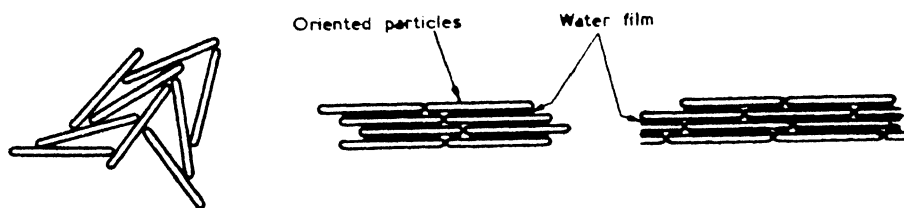
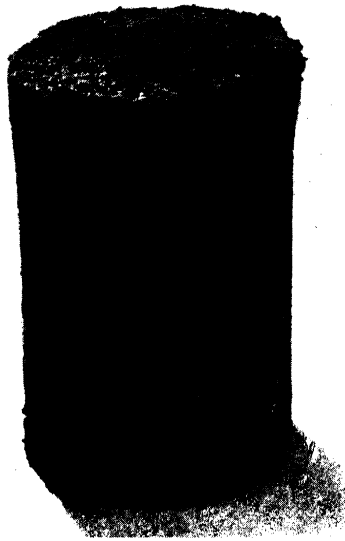
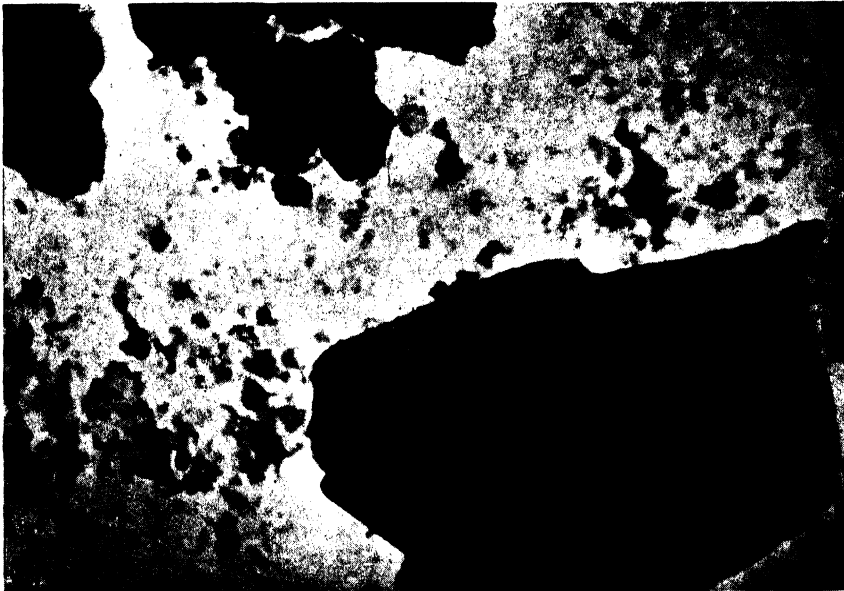



FIG 2·6 ORIENTATION OF PARTICLES AS A CAUSE OF SOIL PLASTICITY (Baver)

2·69 The films of moisture surrounding clay particles are particularly important owing to the large specific surface of the clay fraction, and the relatively large quantity of moisture that is consequently associated with it. Clay particles are said to be " hydrated," that is, the water associated with them is adsorbed on to their surfaces.

2·70 Since the adsorptive forces decrease in intensity as the distance from the particle surface increases, the condition of the water in contact with the particle also changes. According to some workers that nearest to the surface is in a tightly held, solid form, while some distance away in the body of the water, the water is in the true liquid state. At points in between, the water

(A) SPECIMEN OF SOIL-CEMENT MADE FROM SOIL CONTAINING A SOLUBLE SULPHATE

Showing disruption of top surface caused by salt crystallization

(B) ELECTRON MICROSCOPE PHOTOGRAPH OF PARTICLES LESS THAN 0·002 MM EQUIVALENT DIAMETER

From Gault clay

PLATE 2·1

will have characteristics intermediate between those of a solid and of a liquid. The effects of this adsorbed water on suction, swelling and shrinkage are particularly pronounced in clays.

2·71 The small dimensions of the void spaces that can exist between clay particles result in the permeability of clay soils being very low, and hence they are difficult to drain. The impedance offered to the movement of moisture also makes clay soils subject to long-term consolidation effects. Under road conditions in this country, where freezing may occur for relatively short periods, clay soils are not normally susceptible to damage by frost for the same reasons.

2·72 In the previous discussion on the water in soil, reference has been made to ions, such as those of hydrogen, sodium, calcium, etc., which are present in the liquid. These ions may be adsorbed on to the surfaces of the soil particles. The phenomenon is particularly marked with the clay fraction because of the large specific surface, and is a reversible process, i.e. ions already adsorbed can be replaced by others in the surrounding solution. The ions are therefore referred to as exchangeable bases, and the capacity of the soil to bind them is referred to as the base exchange capacity. It is usually expressed in terms of the milligram-equivalents of ion that can be held in 100 gm of the soil. The exchange capacity is relatively constant and is a function of the amount and type of clay present in the soil. It may vary from values of the order of 10 mgm-equivalents for a kaolin clay to values of the order of 100 mgm-equivalents for montmorillonite clays. Thus a soil having an exchange capacity of 30 mgm-equivalents may contain about 0·6 per cent of adsorbed calcium ions by weight.

2·73 The physical properties of a soil may be influenced by the nature of the adsorbed ions. Winterkorn and his co-workers have studied the influence of the presence of different ions on the mechanical and physical properties of clay soils[14] [15] as well as the influence of these ions on soil stabilization processes[16] [17]. The effect of the adsorbed ion is partly a function of its chemical valency and partly of the degree to which it is itself hydrated, since it is also surrounded by an envelope of adsorbed water molecules. When such an ion is adsorbed on to the surface of a clay particle, the water envelope accompanying it has an influence on the water film surrounding the particle, and hence affects the physical properties of the hydrated particle.

TABLE 2·1

EFFECT OF VARIOUS EXCHANGEABLE IONS ON THE PROPERTIES OF PUTNAM SOIL

Property and test	Natural soil	Ion					
		H	Na	K	Mg	Ca	Al
Plasticity							
Liquid limit (%)	64	56	88	53	56	62	60
Plasticity index (%)	41	32	63	25	31	35	34
Shrinkage							
Shrinkage limit (%)	18	16	12	19	12	12	16
Compaction							
Maximum dry density (lb./cu.ft)	88	87	85	90	86	85	84
Optimum moisture content (%)	29	31	31	28	31	32	32

2·74 Table 2·1 shows data due to Winterkorn and Moorman[15] and indicates the extent to which alteration of the adsorbed ion can change the properties of a soil. The test results were obtained by treating samples of the same soil with acid and with different metal salts. The values obtained with the hydrogen- and aluminium-clays were similar, since under acid conditions such as those in the hydrogen-clay, aluminium ions may also be present, being derived from the breakdown of the clay alumino-silicates.

2·75 Baver and Winterkorn have also shown that the affinity of soil for water and its capacity for swelling increase as the base exchange capacity increases[18].

2·76 The nature of the clay fraction is such that the presence of even a relatively small quantity has a marked effect on the properties of a soil of which it is a constituent. Thus, soils which may contain a high proportion of sand particles (70 to 80 per cent) have noticeable cohesive and plastic properties if as little as 10 per cent of clay is present, while a soil need only possess 40 to 50 per cent of clay-size particles to have all the properties of clay in the generally accepted sense.

2·77 METHODS OF PARTICLE-SIZE ANALYSIS. Since each of the particle fractions has a distinctive contribution to make to the properties of the soil, it is of value to determine the extent to which each is present, i.e. to determine the distribution of particle size by means of an analysis of the soil.

2·78 The detailed experimental procedures by which this may be done are given in Chapter 3. Briefly, the methods involve two stages, viz:—

(1) Sieving, for the coarser soil fractions.

(2) Sedimentation analysis, for the finer fractions.

2·79 The sieving methods are similar to those employed in aggregate testing.

2·80 The sedimentation techniques employed are all based on the relationship, developed by Stokes, between the radius and specific gravity of a particle, and the terminal velocity with which it falls through a liquid of known viscosity, under the influence of gravity. For a spherical particle this relationship is:—

$$v = \frac{d}{t} = \frac{2(\gamma_s - \gamma_l)\, g r^2}{9\eta}$$

where v = velocity of the falling particle (cm./sec.)
d = distance (cm.) through which the particle falls in time t sec.
γ_s = density of the particle (gm/cc.)
γ_l = density of the liquid (gm/cc.)
g = acceleration due to gravity (cm./sec./sec.)
r = radius of the particle (cm.)
η = viscosity of the liquid (poise).

2·81 A particle of soil is not usually spherical, but for the purpose of size analysis it is assumed that it falls through the liquid with a velocity equal to that of a sphere of radius r, and 2r is then referred to as the " equivalent particle diameter." The sizes of all the particles dealt with in sedimentation techniques are given as equivalent particle diameters, and not in absolute dimensions.

2·82 From Stokes' equation the radius of the particle (r) and the time (t) it takes to fall a distance (d) are inter-related if the other factors are kept constant. Therefore it can be assumed that all the particles larger than, say, the upper limit of the clay fraction (0·002 mm.) will have passed below a certain depth (d) after a calculated time (t). If a sample of the suspension is taken at this point it will only contain clay particles, and the concentration of the particles in a small element of volume in the suspension at that point will be the same as the concentration in the whole suspension at the beginning of the experiment. The sedimentation analysis of soil therefore involves:—(a) the preparation of a soil suspension of known concentration, (b) the calculation of the times in which particles larger than the size limits of different fractions will have been carried beyond an arbitrarily fixed depth and (c) the determination of the concentration of the particles at this fixed depth, and the expression of this concentration in terms of the original sample used in preparing the suspension.

2·83 When preparing the suspension, it is first necessary to treat a sample of the soil with an oxidizing agent (hydrogen peroxide) to remove organic matter that may bind particles together and cause them to settle as aggregations of particles instead of as individuals. It is also usual at this stage to remove flocculating compounds from the soil, such as those of calcium, by treatment with dilute acid. If such compounds are allowed to remain in the soil they have a tendency to make the finer soil particles aggregate together to form "flocs" in the suspension, which again would settle as larger particles.

2·84 The operations discussed above are referred to as the pre-treatment of the soil. When they are completed the sample will contain hydrogen-saturated clay, i.e. clay in which hydrogen has replaced the exchangeable bases adsorbed on the clay particles. Such hydrogen-clays are difficult to disperse, so that a small quantity of a suitable chemical (sodium oxalate) has to be added to the water with which the suspension is to be made. The sodium ions replace the hydrogen ions on the particles, giving rise to a sodium-clay, which disperses much better in water. Because of this effect the sodium oxalate is referred to as a deflocculating agent. Many such agents have been used for soils, among them ammonia, sodium carbonate and sodium silicate; experience at the Road Research Laboratory, however, has shown that sodium oxalate gives satisfactory results in a very large number of cases.

SUMMARY

2·85 Soil contains three components, viz. air, water and solids. This chapter deals with some of the fundamental physical and chemical factors governing the composition and properties of these components and their constituents, and brief references are made to their influence on the characteristics of the soil as a whole.

2·86 The air content of the soil is of limited interest to the engineer, but it has a bearing on the movement of moisture through the soil in the vapour phase, and on the activities of aerobic bacteria.

2·87 The influence of the water in soil is due partly to its physical characteristics as a liquid, and partly to its chemical nature as a solvent. As a liquid it

affects the cohesion, moisture suction, swelling, shrinkage, and compaction of a soil. The chemical constituents in the water that are of interest to the road engineer include dissolved salts (mainly sulphates), hydrogen ions and exchangeable bases associated with clay soils.

2·88 The solid part of the soil is composed of organic constituents, derived from the vegetable and animal life supported by the surface layers, and inorganic constituents, derived from rocks by the pedogenic processes. The inorganic part is usually composed of mineral particles which are classified into fractions of different particle size, such as sand, silt and clay. The physical properties of a soil are considerably influenced by the distribution of particle sizes within it, and the principles underlying the experimental determination of this size distribution are briefly discussed.

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CHAPTER 3

IDENTIFICATION AND CLASSIFICATION TESTS FOR SOILS

INTRODUCTION

3·1 This chapter describes some of the methods of test in common use for the identification and classification of soils. Many of these methods have been standardized by the British Standards Institution[1] and are employed as routine tests at the Road Research Laboratory. The standard procedures are described in some detail, but other similar methods of test are also reviewed, and the information that can be obtained from them is discussed.

3·2 Many of the tests are used in the different branches of soil work, and procedures similar to those described have been standardized by the American Society for Testing Materials[2] and similar organizations in other countries, In particular, they are used extensively by agricultural research workers, by whom they were first developed.

3·3 The procedures described are:—

(1) Preparation of disturbed soil samples for testing.
(2) Determination of the moisture content of soil.
  (a) Oven-drying method.
  (b) Rapid heating method.
  (c) Pycnometer method.
(3) Determination of the plasticity characteristics of soil.
  (a) Liquid limit.
  (b) Plastic limit.
  (c) Plasticity index.
  (d) Liquidity index.
(4) Determination of the specific gravity of soil particles.
  (a) Laboratory method.
  (b) Pycnometer method.
(5) Determination of the particle-size distribution in soil.
  (a) Pipette method.
  (b) Hydrometer method.
(6) Determination of the shrinkage limit of soil.

3·4 These procedures do not include all the tests used in soil classification systems. The chief omissions are tests for the determination of the field moisture equivalent, the centrifuge moisture equivalent and the compaction characteristics of soil. The compaction test is described in Chapter 9, while the other two tests, which are of little importance, are briefly discussed at the end of this chapter.

3·5 A number of data sheets showing worked examples of the tests are included in an appendix to this chapter.

PREPARATION OF DISTURBED SOIL SAMPLES FOR TESTING

3·6 Samples of soil obtained from the field need to be prepared by a standard method before testing so that reproducible results can be obtained. The usual procedure is to dry the soil and remove the stones, so that the tests are carried out on the finer fractions, that is, the so-called "soil mortar." In the B.S. method the soil is allowed to dry in the air at room temperature; this may take some time, and in practice oven-drying at 105°C. can be used in many cases. With some soils, however, particularly those containing much organic matter and some heavy clays, irreversible changes take place during oven-drying, and anomalous results are obtained in the subsequent tests. In such cases, the soil should be air-dried.

Procedure

3·7 DRYING OF SAMPLE. The sample can be dried, in this country, by leaving the soil spread out on trays in a fairly warm room for three or four days, after which it should be in a state in which it can be crumbled. Alternatively, the soil may be dried in a thermostatically controlled drying oven set at about 105 to 110°C. for about 24 hours (or overnight).

3·8 PULVERIZATION. The dried sample is ground in a mortar by means of a rubber-covered pestle so as to break up the soil aggregates without crushing the individual particles. Alternatively, pulverizing machines, such as a ball-mill, can be used if the particles are not crushed. Pulverization is continued until only individual particles remain when the sample is passed through a No. 7 B.S. sieve. The soil is then separated into two fractions. That retained on the No. 7 B.S. sieve is set aside for use in the coarse particle-size analysis; that passing the sieve is reduced in size by quartering, riffling or some other systematic method of splitting the sample. If the sample has been air-dried, it is usual at this stage to determine the moisture content of the material passing the No. 7 B.S. sieve.

3·9 REDUCTION OF SAMPLE SIZE. Quartering is done by placing the sample of soil in a heap, dividing it into four parts and selecting alternate quadrants. The material selected in this way is then mixed, placed in a fresh heap and the process repeated.

3·10 Riffling requires the use of a riffle box (see Plate 6·5) into which the sample is poured and divided into two halves by a series of alternate chutes. Halving is repeated until a sample of suitable size is obtained.

3·11 ALLOCATION OF SAMPLE. About 150 gm of the portion of the sample so selected is then set aside for use in the fine particle-size analysis. The remainder is separated into two fractions by means of the No. 36 B.S. sieve. That retained on the sieve is discarded; that passing the sieve is reduced by the same method as before to a quantity of about 300 gm and used in the soil plasticity tests. The liquid limit test requires about 100 gm and the plastic limit test about 15 gm; it is useful to keep the remainder in case check tests need to be made.

DETERMINATION OF SOIL MOISTURE CONTENT

Definition

3·12 The moisture content of a soil is defined as the ratio of the weight of water present in the soil to the dry weight of the solid soil particles, and is expressed as a percentage.

Uses of Test

3·13 The effect of moisture is of prime importance in all branches of soil mechanics and, in nearly all soil tests, determinations of moisture content have to be made. Besides these laboratory tests, determinations of natural moisture contents are required to provide information on the moisture conditions of soils in the field.

Review of Methods

3·14 A number of methods of determining soil moisture content both in the laboratory and in the field have been developed. Of these one laboratory method and two field methods have been adopted as British Standards and are described fully here.

3·15 OVEN-DRYING METHOD. The B.S. laboratory method in which the soil is dried in an oven at 105 to 110°C. is the most accurate and should be used whenever possible. In the field, considerations of time (the test takes up to 24 hours) or availability of ovens often preclude its use. However, if the results are not required urgently it is more satisfactory to keep moisture content samples in carefully selected containers, until an oven is available, than to use one of the subsidiary methods.

3·16 SAND-BATH METHOD. In one of the B.S. field methods, the soil is dried on a sand-bath heated over a gas or primus flame. Drying takes from about twenty minutes to an hour according to the soil type, and the method thus provides a fairly rapid determination of moisture content. It suffers, however, from the disadvantage that it tends to give high results when clay or combustible organic matter is present in the soil. When organic soils are strongly heated, as in this method, they tend to decompose, giving a reduced weight of dry soil. Also, when clay particles are heated above 105 to 110°C. the crystalline structure begins to break down and chemically bound water is lost.

3·17 PYCNOMETER METHOD. The second B.S. field method entails the use of a pycnometer, which is in effect a large density bottle. The method involves placing a soil sample in the bottle and filling it with water, and the results depend for their accuracy upon the thoroughness with which air is removed from the soil by shaking. This is difficult with fine-grained soils such as clays, and even if smaller samples are taken to ensure the complete removal of air bubbles, other inaccuracies are introduced. The use of the method should therefore be limited as far as possible to coarse-grained soils such as sands and gravels. The method is used with widespread success on similar materials in connexion with concrete work. The determination only takes about five minutes, which is often an important consideration in the field. On the other hand, the method requires a knowledge of the specific gravity of the soil tested. The method is therefore generally limited to use on occasions when a large number of determinations of moisture content are made upon only one or two soil types.

3·18 ETHYL ALCOHOL METHOD. In a third field method of determining moisture content, the wet soil is weighed, placed in an evaporating dish and saturated with alcohol methylated spirit). The alcohol is ignited and fresh applications made until the heat has driven off all the moisture from the soil, which is then re-weighed. Two applications are usually sufficient and the test requires about ten minutes. The method suffers from the same disadvantages as the sand-bath method as the soil is liable to be " burnt "; it is also sometimes difficult to control the flame.

3·19 DENSITY METHOD. With this method a relationship is obtained between the bulk density and moisture content of a soil by carrying out a compaction test as described in Chapter 9. All subsequent determinations of moisture content are made by carrying out a similar test and measuring the density by weighing or by the use cf a Proctor needle. A description of a Proctor needle is given in Chapter 9. The method is therefore limited to use on occasions when a large number of determinations of moisture content have to be made upon one or two types of soil. The needle method is most useful as a field test on fine-grained cohesive soils, but the weighing method can only be applied to cohesive soils that are in a moisture condition at or near saturation.

3·20 OTHER METHODS. Other methods[3] that have been developed include the use of an electric fan in conjunction with heating coils to blow hot air through a small chamber containing the wet soil. This method is rapid but the finer soil particles tend to be lost. One method makes use of the fact that water reacts with calcium carbide to form acetylene gas. Carbide is mixed with the wet soil in an airtight container and the pressure of gas generated is measured on a dial. This takes less than five minutes to perform but the instrument has to be calibrated for each soil, since there is an increasing negative error with increasing clay content.

Procedure—B.S. Laboratory Method

3·21 OVEN-DRYING METHOD. For cohesive soils the apparatus required is either a small glass weighing bottle with an airtight stopper, or a small non-corrodible metal container with a tight-fitting lid, such as an ointment tin (Plate 3·1A). The tare weight (W_1) should be known in either case and can be determined on a balance capable of weighing up to 500 gm accurately to 0·01 gm.

3·22 For stony soils a larger container is needed, e.g., a 1-lb. size cylindrical tin with a lever-lid (Plate 3·1A). For both types of soil a drying oven capable of maintaining a controlled temperature of 105 to 110°C. is required.

3·23 A sample of cohesive soil of about 30 gm weight (or 250 to 300 gm of stony soil) is placed in the appropriate container and the whole weighed (W_2). The lid or stopper is then removed and the container is placed in the oven and dried for 16 to 24 hours. This period is usually sufficient; if particular accuracy is required heating and weighing of the dried soil should be repeated until a constant weight is obtained.

3·24 On removal from the oven, the container and contents are allowed to cool in a desiccator, after which the lid is firmly replaced and the whole weighed (W_3). If a desiccator is not available, the dried sample should be cooled in its container with the lid in place.

3·25 The moisture content (m) is calculated as a percentage of the dry weight from the formula:—

$$m = \frac{W_2 - W_3}{W_3 - W_1} \times 100 \text{ (per cent)}$$

The result is usually expressed to the nearest whole number.

Procedure—Field Methods

3·26 SAND-BATH METHOD. The moisture content is determined, in much the same way as in the previous method, except that instead of being oven-dried the sample is placed in its container on a sand-bath which is heated by a primus stove, bunsen burners, or other suitable means. For this test a metal container should be used and the soil can be occasionally disturbed with a spatula during heating to assist evaporation. The test may take up to an hour according to the soil type; thus, if a number of tests are being performed on one soil type it is worth while determining the minimum drying period at the start by carrying out several tests using different periods.

3·27 PYCNOMETER METHOD. The pycnometer (Plate 3·1B) consists of a screw-top glass container such as a 2-lb. fruit jar, with a specially made conical top having a sharp-edged hole of about ¼-in. diameter at its apex. To prevent leakage a rubber or fibre washer is placed between the lid and the jar. If this washer is soft, the screw-top and the jar must be marked so that the top is screwed down to the same position every time and the volume of the pycnometer thus kept constant. Other apparatus required for the test includes a 5-kgm balance accurate to 1 gm, a glass rod about 12 in. long and a thermometer.

3·28 The pycnometer is first used to determine the specific gravity of the soil particles (G_s). For this an oven-dried sample of the soil in question is required: the method used in this determination is described later.

3·29 When the specific gravity of soil particles has been determined, the pycnometer is filled with water at a temperature of between 15 and 25°C. until the level is flush with the hole in the screw-top. The outside of the pycnometer is then wiped dry and its weight (W_1) recorded to the nearest gm. A sample of the soil of about 400 to 500 gm is then weighed out (W_2), the pycnometer half emptied, the screw-top removed and the sample placed inside. This soil-water mixture is thoroughly stirred with the glass rod to remove air entrapped in the soil, the screw-top replaced and the pycnometer again filled with water. Any remaining air is removed by shaking the pycnometer, holding one finger over the hole at the top. This may tend to form a froth which collects under the top; if so, it should be carefully removed and the pycnometer topped up with water. The outside is then dried and the weight (W_3) recorded.

3·30 The moisture content is then given by the formula:—

$$m = \left\{ \frac{W_2 (G_s - 1)}{G_s (W_3 - W_1)} - 1 \right\} \times 100 \text{ (per cent)}$$

Fig. 3·1 shows a chart which may be used instead of this calculation if the weight of wet sample taken is always 500 gm.

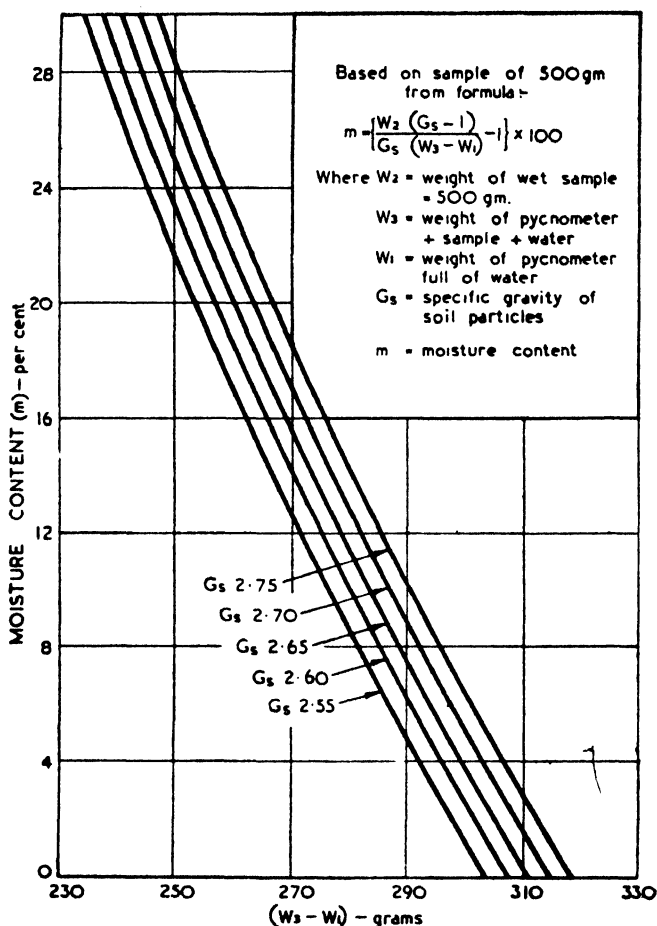



FIG. 3·1 CHART FOR THE CALCULATION OF MOISTURE CONTENT BY THE PYCNOMETER METHOD

DETERMINATION OF SOIL PLASTICITY

3·31 Plasticity is a major characteristic of the so-called " cohesive " soils, that is, soils containing an appreciable proportion of clay particles. It is the property that enables a material to suffer deformation without noticeable elastic recovery and without cracking or crumbling.

3·32 Many methods, mainly arbitrary, exist for determining the plasticity of soils. The tests due to Atterberg[4], originally developed for agricultural work, have won wide acceptance in the field of soil engineering and have been included in a number of soil classification systems and specifications. The classification system with which they are at present most intimately associated is that due to A. Casagrande[5]. (See also Chapter 4.)

Definitions

3·33 The two tests concerned attempt to fix the moisture contents at which a clay soil passes from the solid to the plastic and the plastic to the liquid states, and are consequently referred to as the " plastic limit " and the " liquid limit " tests. Briefly, the liquid limit of a soil is defined as the moisture content at which the soil is sufficiently fluid to flow a specified amount when lightly jarred 25 times in a standard apparatus. The plastic limit is defined as that moisture content at which a thread of soil can be rolled without breaking until it is only ⅛ in. in diameter. The numerical difference between the liquid and plastic limits is termed the " plasticity index " and indicates the magnitude of the range of the moisture contents over which the soil is in a plastic condition as defined by the tests.

Implications of the Liquid and Plastic Limit Tests

3·34 SOIL CLASSIFICATION. The liquid and plastic limits are both dependent on the amount and type of clay in a soil, but the plasticity index is generally only dependent on the amount of clay present. It follows that information regarding the type of clay in the soil may be obtained by considering the plasticity index in relation to the liquid limit, while the amount of clay present may be deduced, within broad limits, from the value of the plasticity index. These facts form the basis for the soil classification systems for cohesive soils based on the plasticity tests.

3·35 OTHER SOIL PROPERTIES INDICATED. Besides their use for identification, the plasticity tests give information concerning the cohesive properties of a soil and the amount of capillary water which it can hold. Further characteristics that are indicated directly by these tests have been summarized as follows by Casagrande:—

Characteristic	Comparing soils of equal LL with PI increasing	Comparing soils of equal PI with LL increasing
Compressibility	About the same	Increases
Permeability	Decreases	Increases
Rate of volume change	Increases	—
Toughness near PL	Increases	Decreases
Dry strength	Increases	Decreases

3·36 It should be remembered when dealing with the results of plasticity tests that they are sometimes affected if the soil has been oven-dried. The liquid and plastic limits of organic clays determined from oven-dried samples are much lower than the limits of the same soil that has not been oven-dried. Oven-drying of an inorganic soil results only in small changes in its index properties.

Determination of the Liquid Limit of Soils

3·37 APPARATUS. The B.S. apparatus required is shown in Plate 3·2A and Fig. 3·2. In this device a brass cup is raised 1 cm. above a flat base and then dropped, by rotating a handle. A B.S. grooving tool is also required; this usually has a gauge which is used to check that the cup is adjusted to give a drop of exactly 1 cm. An alternative special grooving tool may be used for

more sandy soils, when the sides of the grooves tend to tear. A spatula and either a glass plate 2 ft square or a 14-cm. diameter evaporating dish are needed for mixing the soil sample with water.

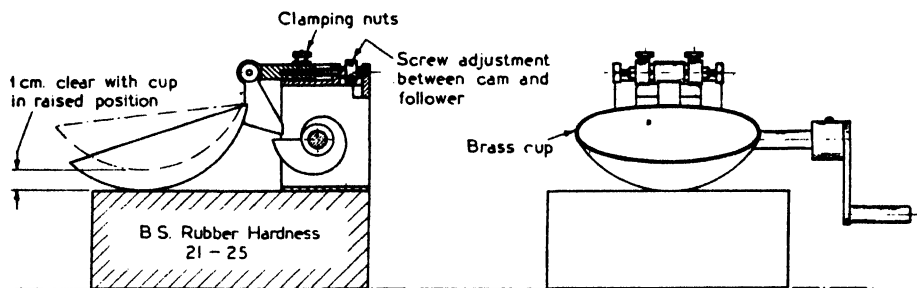
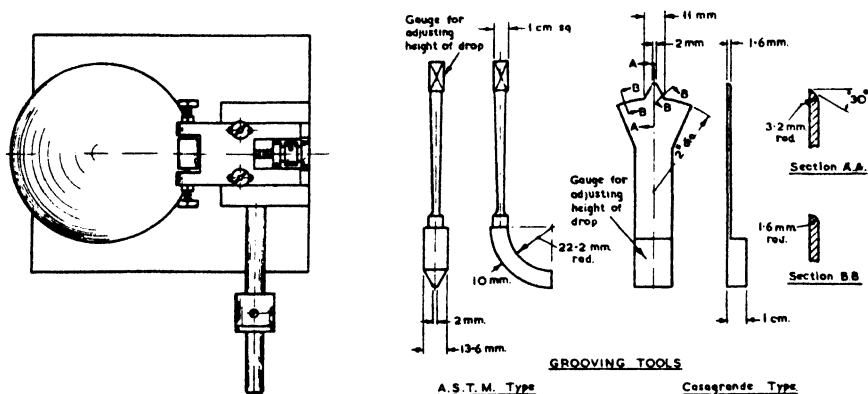



FIG. 3·2 LIQUID LIMIT DEVICE AND TOOLS

3·38 PROCEDURE. A sample of at least 100 gm is taken from the material passing the No. 36 B.S. sieve. (See paras. 3·6 to 3·11.) It is placed on the glass plate and thoroughly mixed with distilled water, using the spatula, until the mass becomes a thick paste of a putty-like consistence. A portion of the mixture is placed in the cup and levelled with the spatula to a maximum depth of 1 cm. The grooving tool is then used to divide the soil pat along the diameter through the centre of the hinge. By turning the handle of the machine at a rate of about 2 rotations per second, the cup is lifted and dropped repeatedly until the two parts of the soil sample come into contact at the bottom of the groove along a distance of ½ in. The number of blows which have been given is noted.

3·39 A small quantity of soil from the portions of the sample that have just flowed together as well as some of the soil removed by the grooving tool is then placed in a container and the moisture content determined. The rest of the sample is then re-mixed with a small addition of distilled water and the

operation repeated. Altogether the test should be done at least four times, the moisture content being chosen to give results that are distributed evenly between 10 and 50 blows of the machine.

3·40 INTERPRETATION OF RESULTS. The results are plotted on a "flow curve" as shown in the Appendix to this chapter (Data Sheet 3·A). This relates the moisture content to the corresponding number of blows and is plotted on a semi-logarithmic chart with the percentage moisture contents as abscissae on the arithmetical scale and the number of blows as ordinates on the logarithmic scale. A straight line is drawn through the points plotted and from it the moisture content corresponding to 25 blows is read off as the liquid limit (LL) of the soil. It is usually expressed to the nearest whole number.

Determination of the Plastic Limit of Soils

3·41 PROCEDURE. In this test a 15-gm sample, prepared in the manner previously described, is used. It is thoroughly mixed with distilled water on a glass plate until it is plastic enough to be rolled into a ball. (It is often convenient to allow soil used in the liquid limit test to dry in the air until it is in this condition.) The ball of soil is then rolled between the hand and the glass plate as in Plate 3·2B so as to form the soil mass into a thread. When the diameter of the thread becomes less than $\frac{1}{8}$ in. the soil is kneaded together and rolled out again. In this way the water in the sample is evaporated by the heat of the hand until the soil just ceases to be plastic and crumbles. When crumbling of the thread occurs at a diameter of $\frac{1}{8}$ in., the portions of crumbled soil are gathered together and placed in a container for a moisture content determination. Duplicate determinations are made and the average value of these moisture contents is taken as the plastic limit (PL) of the soil. It is expressed to the nearest whole number.

Calculation of the Plasticity Index of Soils

3·42 PROCEDURE. The plasticity index is calculated from the formula:—

$$\text{Plasticity Index} = \text{Liquid Limit} - \text{Plastic Limit}$$

or

$$PI = LL - PL$$

3·43 When either the liquid limit or plastic limit cannot be determined the plasticity index is reported as N.P. (non-plastic). When the plastic limit is equal to or greater than the liquid limit the plasticity index is reported as 0 (zero).

Calculation of the Liquidity Index of Soils

3·44 DEFINITION. The liquidity index of a soil is defined as the moisture content of the soil in excess of the plastic limit, expressed as a percentage of the plasticity index. It merely describes the moisture condition of a soil with respect to its index limits and is of no use for classification purposes. It shows in what part of its plastic range a given sample of soil lies. This is sometimes convenient in assessing the condition of a soil at its natural moisture content in the field.

3·45 PROCEDURE. The natural moisture content (m), liquid and plastic limits are determined in the usual way and the liquidity index calculated from the formula:—

$$LI = \frac{m - PL}{LL - PL} \times 100 \text{ (per cent).}$$

DETERMINATION OF THE SPECIFIC GRAVITY OF SOIL PARTICLES

3·46 Although in general a knowledge of the specific gravity of soil particles is of little use in the identification of soils a high value indicates the presence of unusual minerals.

3·47 Determination of the specific gravity of the soil particles are required:—

(1) In the calculation of the voids ratio of specimens of soil.

(2) In the determination of the moisture content of the soil by the pycnometer method.

(3) In particle-size analysis.

Methods of Determination

3·48 Both the methods of determining specific gravity to be described depend for their accuracy upon the thoroughness with which air is extracted from a sample of the soil immersed in water. In the laboratory method, using the usual type of density bottle, the air is extracted by a vacuum pump and a fairly accurate result is obtained. In the field method, using a pycnometer, however, the air is removed simply by shaking and results are less accurate. This applies particularly to soils with a large clay fraction, from which it is more difficult to remove the entrapped air.

Laboratory Method using a Density Bottle

3·49 APPARATUS. The apparatus required is:—

(1) A 50-ml. density bottle with a perforated stopper (B.S. No. 733).

(2) A constant-temperature water bath at 20°C. (Plate 3·3A).

(3) A vacuum desiccator.

(4) A drying oven at 105 to 110°C.

(5) A balance to weigh 100 gm to an accuracy of 0·001 gm.

3·50 PROCEDURE. The density bottle (including stopper) is first dried in the oven and weighed accurately to the nearest 0·001 gm (W_1). About 25 gm of oven-dried soil are cooled in a desiccator and then placed in the density bottle. The complete bottle and contents are weighed (W_2). Next the bottle is half-filled with distilled water and, with the stopper removed, placed in the desiccator, which is gradually evacuated. Care should be taken to see that the air entrapped in the soil does not bubble too violently in case small drops of suspension are lost through the mouth of the bottle. When no further air is released, the bottle is removed from the desiccator and filled with distilled water, the stopper and cover replaced, and the bottle immersed in the constant temperature bath until it reaches a temperature of 20°C. (About 1 hour is

usually sufficient.) The stopper is then removed and any decrease in volume of the distilled water is made good by adding more and the whole wiped dry and weighed (W_3).

3·51 EXPRESSION OF RESULTS. The specific gravity of the soil particles is then given by:—

$$\text{Specific Gravity} = \frac{W_2 - W_1}{50 - W_3 + W_2}$$

In certain cases a liquid other than water has been found suitable for this test, i.e. alcohol, paraffin or benzene. If one of these is used the specific gravity is given by:—

$$\text{Specific Gravity} = \frac{(W_2 - W_1)\,G_l}{50\,G_l - W_3 + W_2}$$

where G_l = specific gravity of the liquid used, at 20°C. It is usual to make duplicate determinations.

Field Method using a Pycnometer

3·52 PROCEDURE. This is substantially the same as the laboratory method, except that a field pycnometer (see Plate 3·1B) is employed instead of an accurately calibrated density bottle.

3·53 The pycnometer should first be dried carefully and weighed (W_1) after which it is filled with clean tap water and weighed again (W_2). The water is then emptied and the bottle again dried. About 400 to 500 gm of oven-dried soil are introduced, and the pycnometer and contents again weighed (W_3). Water is then added to the soil, which is stirred with a glass rod during the process to allow entrapped air to be released. Finally, the jar is filled completely with water, dried on the outside and weighed again (W_4).

3·54 EXPRESSION OF RESULTS. The specific gravity of the soil particles can then be calculated from the formula:—

$$\text{Specific Gravity} = \frac{W_3 - W_1}{W_2 - W_1 + W_3 - W_4}$$

Fuller details of the use of the pycnometer are given in para. 3·17.

DETERMINATION OF THE PARTICLE-SIZE DISTRIBUTION IN SOIL

3·55 The particle-size distribution in a soil is found by particle-size analysis or, as it is often called, "mechanical analysis." An analysis of this kind expresses quantitatively the proportions by weight of the various sizes of particles present in the soil. A very wide range of particle sizes is encountered in soil and it is convenient to divide them into fractions having substantially different properties. Thus "gravel," "sand," "silt" and "clay" fractions are recognized as containing particles of decreasing magnitude. The actual dimensions of the particles are usually given in terms of "equivalent particle diameters" and size fractions can therefore be specified as lying between

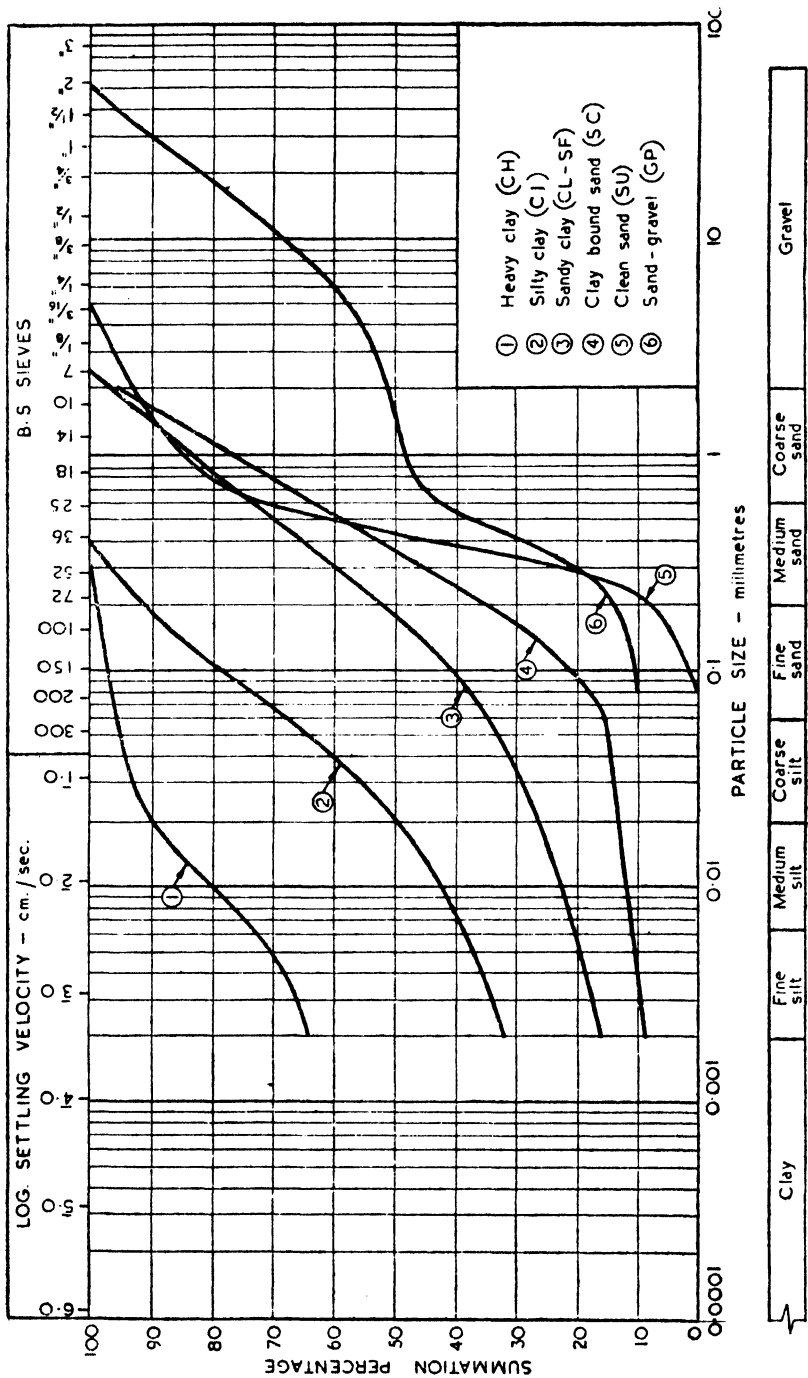



FIG. 3·3 TYPICAL PARTICLE-SIZE DISTRIBUTION CURVES FOR DIFFERENT SOILS

certain limits of particle diameters. In the British Standard system the various fractions have the following limits of equivalent particle diameters:—

Gravel	60 — 2·0 mm.	equivalent particle diameter
Sand	2·0 — 0·06 mm.	,, ,, ,,
Silt	0·06 — 0·002 mm.	,, ,, ,,
Clay	< 0·002 mm.	,, ,, ,,

3·56 In addition to this tabular method of specifying the particle-size distribution in a soil, it is also possible to make a graphical representation of it in the form of a particle-size distribution curve. In such a graph, the cumulative percentages finer than various equivalent particle sizes are plotted against these sizes, the latter being on a logarithmic scale. Fig. 3·3 shows the particle-size distribution curves of several soils plotted in this way. The results of particle-size analyses are widely used in soil classification and in other soil studies. Thus, the Casagrande classification[5] requires a particle-size analysis to be made on coarse-grained soils, and the U.S. Public Roads system[6] employs it for both coarse- and fine-grained soils. In addition, textural charts of the triangular type (see Fig. 4·5) are often used as a means of describing soils. Particle-size distribution curves are also used in conjunction with graphically expressed particle-size limits to determine the suitability of soils for earth road construction.

Methods of Determination

3·57 Both in the laboratory and in the field, particle-size analysis is carried out by combining sieving and sedimentation methods. When the gravel fraction has been removed by sieving on the No. 7 B.S. sieve, it is subjected to a normal particle-size analysis with sieves. A sample of the material passing the No. 7 B.S. sieve is first treated with hydrogen peroxide to remove organic matter, and with hydrochloric acid to remove carbonates and gypsum, since these substances may give rise to false results in the sedimentation analysis which follows. The soil is then dispersed in a dilute aqueous solution of sodium oxalate, which acts as a deflocculating agent. The particles are then allowed to settle. Particles of different size have different settling velocities in accordance with Stokes' Law, from which, when the velocity is measured, the size can be computed on the assumption that the particles are spherical in shape. If, after a given time from the initial dispersal, samples are taken from a given depth below the surface, the liquid will contain only those particles whose velocities have been insufficient to carry them further. In the B.S. laboratory method samples are taken at various time intervals by means of a pipette and when the samples have been dried and weighed the percentages of different particle sizes are determined.

3·58 In the field method of particle-size analysis the rate of sedimentation is determined from the rate of decrease in density of the upper part of the liquid as the larger particles settle out. The density is measured by a hydrometer.

3·59 The pipette method of particle-size analysis is specified by the British Standards Institution as a primary standard method for laboratory purposes. The apparatus required is, however, expensive and delicate and is not convenient for running control tests during construction or for field work. On

the other hand the hydrometer method has been shown to give results which for all practical purposes are the same when carefully carried out, and the apparatus required is simple and convenient to operate under field conditions.

3·60 Neither of the methods of analysis described here is suitable for use with predominantly chalky soils, owing to the removal of the calcium compounds during the pre-treatment of the soil with hydrochloric acid. However, the experimental details described have been found suitable for a very large number of soils.

Laboratory Method using a Pipette

3·61 APPARATUS. The apparatus and reagents required for both coarse and fine analyses are:—

(1) A sampling pipette of the type illustrated in Fig. 3·4 fitted with a pressure and suction inlet, and having a capacity of about 10 ml. The exact capacity of the pipette V_P including the hole in the tap, must be determined accurately by weighing the amount of water required to fill it. The pipette should be mounted above a constant-temperature bath in such a way that it can be lowered into a boiling tube immersed in the latter (Plate 3·3A).

(2) A glass boiling tube, 5 cm. in diameter and approximately 34 cm. long, graduated at 500 ml. volume.

(3) A constant temperature water bath maintained at 25°C. to an accuracy of $\pm$ 0·1°C. (Plate 3·3A).

(4) Glass weighing bottles, about 25 mm. in diameter and 55 mm. high fitted with ground glass stoppers as in Plate 3.1A.

(5) A high-speed mechanical stirrer, revolving at not less than 5,000 r.p.m. and a stirring cup fitted with wire baffles (Plate 3·3B). (An electrically driven milk mixer is very suitable.)

(6) B.S. sieves of the following sizes:—$1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., $\frac{3}{16}$-in. and No. 25, No. 72 and No. 200.

(7) Balances for weighing up to 100 gm to an accuracy of 0·001 gm and up to 5 kgm to an accuracy of 1 gm.

(8) A thermostatically controlled drying oven set at 105 to 110°C.

(9) A stop-watch, desiccator, evaporating dish, filter funnel, wash-bottle, etc.

(10) A.R. grade sodium oxalate, 6 per cent hydrogen peroxide and 0·2 N. hydrochloric acid*.

3·62 PROCEDURE—COARSE ANALYSIS. The gravel fraction retained on the No. 7 B.S. sieve during the preparation of the original sample is separated into fractions of different particle sizes by means of $1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in. and $\frac{3}{16}$-in. B.S. sieves and the portion retained on each sieve is weighed. Sieving should be done by means of a lateral and vertical movement of the sieve accompanied by a slight jarring action (as described in B.S. 812). Particles smaller than $\frac{3}{4}$-in. should not be helped by hand.

*6 per cent hydrogen peroxide is available commercially in the form of a so-called "20-volume" solution, that is it will yield 20 times its own volume of oxygen when decomposed. 0·2 N. hydrochloric acid can be prepared by carefully diluting 17 ml. of the concentrated acid to 1,000 ml. with distilled water.

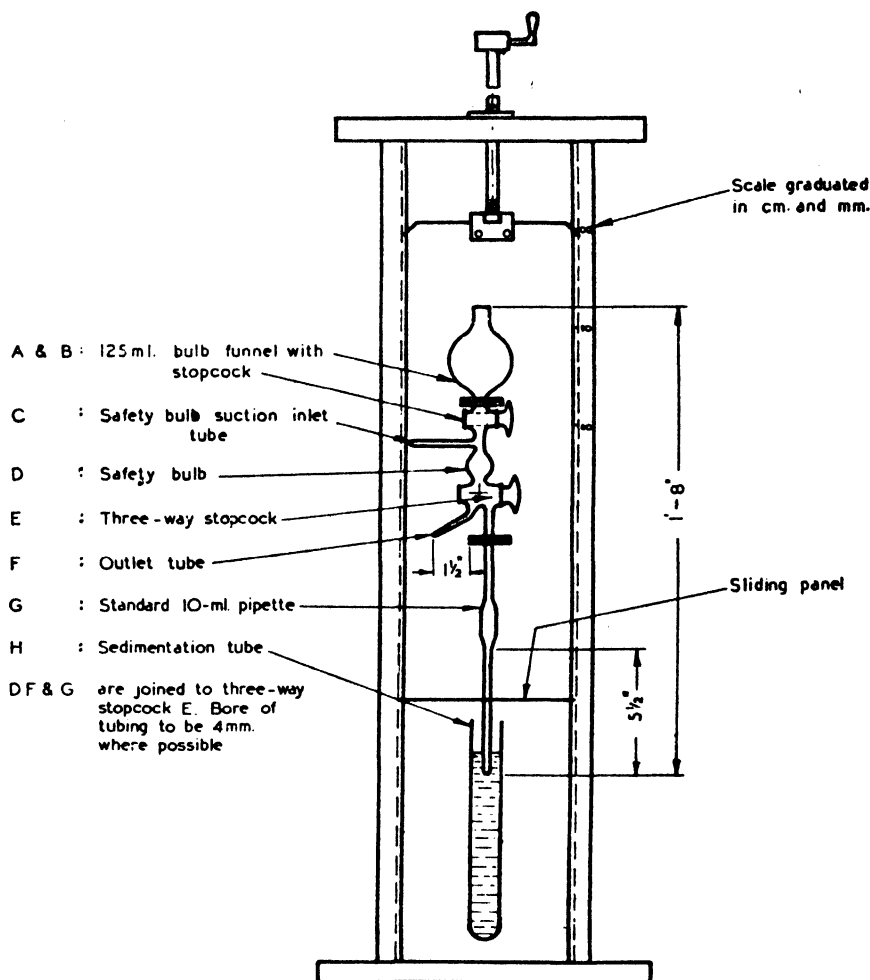



FIG. 3·4 DIAGRAMMATIC ARRANGEMENT OF 10-ML. SAMPLING PIPETTE FOR SEDIMENTATION ANALYSIS

3·63 PROCEDURE—FINE ANALYSIS—PRETREATMENT. About 100 gm of the prepared sample which passes the No. 7 B.S. sieve are weighed and placed in an evaporating dish and about 100 ml. of a 20-volume solution of hydrogen peroxide are added. The mixture is warmed gently to a temperature of not more than 60°C. and stirred until no further evolution of gas takes place. With very organic soils this process may go on for several days and further applications of peroxide may be required. Finally, any excess peroxide is decomposed by boiling the mixture for a few minutes.

3·64 When the mixture has cooled, about 100 ml. of 0·2 N. hydrochloric acid are added to remove the calcium compounds. If these are excessive, further applications of acid may be needed. When the solution gives an acid reaction to litmus, it is filtered and washed with warm water (Plate 3·4A) until the filtrate

shows no acid reaction to litmus. The damp soil on the filter paper is transferred back to the evaporating dish using a jet of distilled water from a wash-bottle, after which it is dried in the oven and weighed. It is often useful to record the loss in weight as the " loss due to pretreatment."

3·65 DISPERSION. About 10 gm ot the pretreated sample are weighed out accurately and placed in a clean evaporating dish and covered with 100 ml. of distilled water. To this are added 50 ml. of a solution containing 8 gm of sodium oxalate per litre. The mixture is warmed for 10 minutes and then transferred to the dispersing cup of the high-speed stirrer. This is done by means of a wash-bottle and care should be taken to see that the amount of water used in doing so is less than 150 ml. The soil suspension is then stirred by the machine for 15 minutes.

3·66 The suspension is next carefully removed from the cup and washed through a No. 200 B.S. sieve, again using not more than 150 ml. of water from the wash-bottle in the operation. The suspension passing the sieve is transferred to the graduated boiling tube and the volume of liquid made up to 500 ml. with distilled water.

3·67 The material retained on the No. 200 B.S. sieve is dried by placing the sieve in the oven. When dry this material is re-sieved (in the dry state) on the Nos. 25, 72 and 200 B.S. sieves, and the fractions retained on each sieve are transferred to suitable bottles and weighed. These weights are recorded as the weights of coarse, medium and fine sand (W_{cs}, W_{ms}, W_{fs}) in the sample. (Dry sieving is used in order to give a more accurate determination of the sand fraction, since with wet sieving the surface tension of water held between the sieve meshes tends to cause particles slightly smaller than the apertures of the No. 200 B.S. sieve to be retained on it.)

3·68 SEDIMENTATION. The boiling tube containing the 500 ml. of suspension should now be immersed up to the 500-ml. mark in the constant-temperature bath for one hour. The suspension should then be at a temperature of 25°C.; it is shaken up thoroughly by removing the tube from the bath, placing a bung in the top and inverting it end over end several times. The bung is removed and the boiling tube replaced in the bath. This is taken to be the zero settling time and the stop-watch is started.

3·69 When sedimentation has been in progress for 4 min. 8 sec. a 10-ml. sample (V_p) of the suspension is taken from a depth of 10 cm. by means of the sampling pipette. The sample is transferred to a weighing bottle and dried in the oven. The weight of solid material in the sample is then determined by weighing. The weight of solid matter in 500 ml. of the suspension (W_1) is calculated as follows:—

$$W_1 = \frac{\text{Wt. of solid material in sample}}{V_p} \times 500 \text{ (gm)}$$

where V_p is the volume of the pipette.

3·70 This procedure is repeated at sampling times of 46 min. and 6 hours 54 min. after the commencement of sedimentation, the boiling tube being shaken afresh in both cases. The weight of solid material in 500 ml. of suspension is determined as before for each sample and the weights W_2 and W_3 obtained.

3·71 The actual method of sampling is important. The pipette should be lowered into the suspension at a slow rate about 20 sec. before the sample is due to be taken, in order to avoid undue disturbance. At the appropriate time the tap on the pipette is opened, the suspension is drawn into the pipette until it is full and the tap closed again. The time taken for the actual sampling operation should be 20 sec. to avoid inaccuracies due to disturbance. An aspirator can be fitted to the pipette in order to ensure that the suspension is drawn in at a uniform rate.

3·72 Any surplus suspension drawn up past the tap can be drained away by turning the lower tap to connect it with the outlet tube. When this surplus has been flushed out with distilled water, the tap is reversed so as to open the pipette and the sample drained into a weighing bottle held below. The pipette should then be flushed with distilled water so as to transfer any particles of soil adhering to the inside of it into the weighing bottle.

3·73 CALCULATION. In the coarse analysis the percentage by weight of material retained on the $1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., and $\frac{3}{16}$-in. B.S. sieves is calculated as well as the percentage passing the $\frac{3}{16}$-in. B.S. sieve. From these values the cumulative percentages by weight of the total sample passing each sieve are calculated.

3·74 In the analysis of fine material, the percentage of coarse, medium and fine sand in the sample are calculated from the formulæ:—

$$\text{Coarse sand} \quad (2{\cdot}0 \text{ mm.} - 0{\cdot}6 \text{ mm.}) = \frac{W_{cs}}{W} \times 100 \text{ (per cent)}$$

$$\text{Medium sand} \quad (0{\cdot}6 \text{ mm.} - 0{\cdot}2 \text{ mm.}) = \frac{W_{ms}}{W} \times 100 \text{ (per cent)}$$

$$\text{Fine sand} \quad (0{\cdot}2 \text{ mm.} - 0{\cdot}06 \text{ mm.}) = \frac{W_{fs}}{W} \times 100 \text{ (per cent)}$$

where W is the total weight of the original dispersed soil sample.

3·75 The percentages of fine and medium silt, and clay are calculated from the results of the sedimentation analysis by means of the formulæ:—

$$\text{Medium silt} \quad (0{\cdot}02 \text{ mm.} - 0{\cdot}006 \text{ mm.}) = \frac{W_1 - W_2}{W} \times 100 \text{ (per cent)}$$

$$\text{Fine silt} \quad (0{\cdot}006 \text{ mm.} - 0{\cdot}002 \text{ mm.}) = \frac{W_2 - W_3}{W} \times 100 \text{ (per cent)}$$

$$\text{Clay} \quad (<0{\cdot}002 \text{ mm.}) = \frac{W_3 - 0{\cdot}400}{W} \times 100 \text{ (per cent)}$$

3·76 The factor 0·400 in the last formula is to allow for the amount of solid sodium oxalate that will be present in the dried pipette sample of the clay fraction.

3·77 The percentage of coarse silt (0·06 mm. — 0·02 mm.) in the original sample is obtained by subtracting the sum of the percentages of all the fractions given above from 100.

3·78 When reporting the results any material coarser than 2 mm. diameter is referred to as gravel. Although the aperture of the No. 7 B.S. sieve is 2·4 mm. there is in practice only a very slight error in using this sieve for separating gravel from coarse sand particles.

3·79 PARTICLE-SIZE DISTRIBUTION CURVE. The results of the coarse and fine analysis can be plotted on a semi-logarithmic graph as in Fig. 3·3. The percentages of various fractions are expressed to the nearest 1 per cent and plotted against the corresponding grain size. Alternatively, the results may be given in tabular form, as the percentages of gravel, sand, silt and clay in the original pretreated soil sample.

Field Method using Hydrometers

3·80 APPARATUS. The apparatus and reagents required are:—

(1) Long- and short-stem hydrometers of the type shown in Fig. 3·5.

(2) A 1000-ml. measuring cylinder.

(3) A 5-in. diameter porcelain mortar and a suitable rubber covered pestle.

(4) B.S. sieves of the following sizes:— $1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., and $\frac{3}{16}$-in., and No. 25, No. 72, and No. 200.

(5) Balances capable of weighing up to 500 gm to an accuracy of 0·01 gm and up to 5 kgm to an accuracy of 1 gm.

(6) A drying oven set at 105 to 110°C.

(7) A stop-watch, desiccator, evaporating dish, filter funnel and paper, wash-bottle, etc.

(8) A.R. grade sodium oxalate, 6 per cent hydrogen peroxide solution and 0·2 N. hydrochloric acid. (See footnote on p. 42.)

3·81 CALIBRATION OF HYDROMETERS. Prior to the sedimentation analysis, each hydrometer requires to be calibrated. This can be done in the following manner.

(1) The volume of the hydrometer (V_h) is measured, either by partly filling the 1000-ml. measuring cylinder and observing the increase in volume of the water when a hydrometer is immersed in it, or by weighing the hydrometer, since the weight of the hydrometer in grams is approximately equal to its volume in cubic centimetres. The stem volume which is included by these methods may be neglected.

(2) The sectional area (A) of the 1000-ml. measuring cylinder is found by measuring the distance between two graduations (say 0 and 900) and dividing this distance into the volume included between the graduations.

(3) The distances from the lowest calibration mark on the stem of the hydrometer to each of the other major calibration marks (R_h) are measured.

(4) The distance from the neck of the bulb to the nearest calibration mark is measured.

(5) The height of the bulb (h), i.e. the distance from the neck to the bottom of the bulb, is measured.

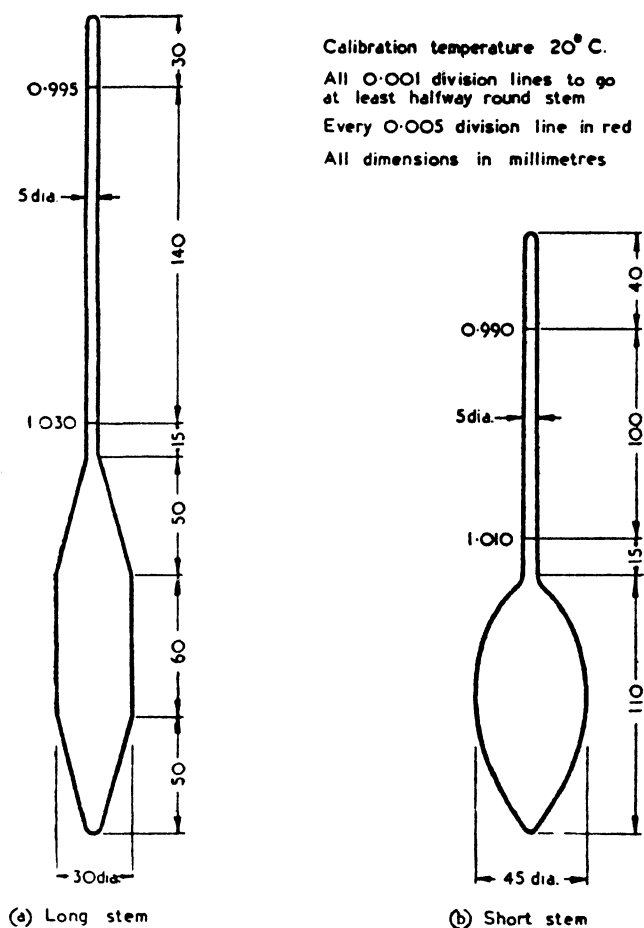



FIG. 3·5 HYDROMETERS
for the determination of particle-size distribution in soil

3·82 If H_1 is taken as the distance corresponding to a reading R_h, and as equal to the sum of the distances measured in (3) and (4), the effective depth H_r of the centre of volume of the hydrometer corresponding to each of the major calibration marks R_h is given by:—

$$H_r = H_1 + \frac{1}{2}\left(h - \frac{V_h}{A}\right)$$

The relationship between H_r and R_h is thus plotted as a smooth curve from which a scale of R_h values can be constructed on the nomographic chart used for the solution of Stokes' Law. (See Chart 3·C in the appendix to this chapter).

3·83 MENISCUS CORRECTION. Since soil suspensions are opaque the true reading of the hydrometer at the bottom of the meniscus of the liquid cannot be obtained. In order to read the hydrometer at the top of the meniscus, a

meniscus correction must be made. To do this the measuring cylinder is partly filled with water and the hydrometer inserted. Then readings on the hydrometer are taken where it intercepts the top and bottom of the meniscus and the difference is taken as the correction (C_m).

3·84 PROCEDURE—COARSE ANALYSIS. For the coarse particle-size analysis the procedure is identical with that already described for the pipette method.

3·85 PROCEDURE—FINE ANALYSIS. Pretreatment is carried out in the same way as in the pipette method, but between 50 and 100 gm of the oven-dry powered sample are weighed out for use in the fine analysis. To this are added 100 ml. of a solution containing 8 gm of sodium oxalate per litre. It is advisable to take a 50-gm sample when the soil is a clay, and 100-gm if it is a sandy soil.

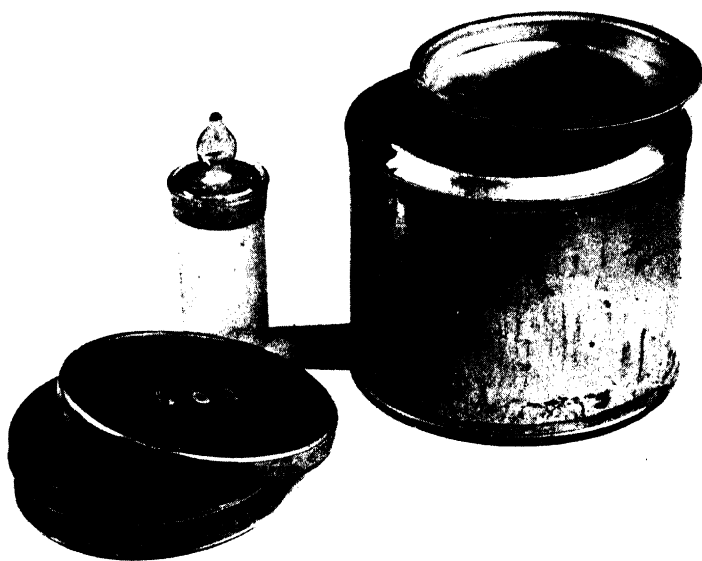
3·86 The mixture is then placed in the mortar and ground vigorously with a rubber pestle. The suspension is allowed to settle for 2 min. and decanted through a No. 200 B.S. sieve into a receiver. More water is added to the mortar, and pestling and decantation are repeated until the decanted liquid is clear.

3·87 The next operation is to transfer the suspension and the residue in the mortar to the No. 200 B.S. sieve by means of a wash-bottle. The suspension passing the sieve is then transferred to the measuring cylinder and made up to exactly 1000 ml. with distilled water.

3·88 The material retained on the sieve is dried in the oven and re-sieved on the Nos. 25, 72 and 200 B.S. sieves to obtain the weights of coarse, medium and fine sand, as in the pipette analysis.

3·89 The measuring cylinder is now shaken vigorously (using a bung in the top of the cylinder). When this is done it is allowed to stand and the stop-watch is started. The long-stemmed hydrometer is carefully inserted and the first reading taken after a period of ½ min. Further readings are taken at 1 and 2 min. and the hydrometer is removed. Insertion and withdrawal of the hydrometer should be done carefully, taking about 10 sec. over each operation, so as to avoid unnecessary disturbance. After each removal the hydrometer should be wiped dry with a clean rag. The stem should be kept perfectly clean, otherwise the meniscus formed may not be fully developed, resulting in inaccurate readings.

3·90 Further readings should be taken after periods of 4, 8, 15 and 30 min. and 1, 2 and 4 hours. Subsequently, readings may be taken once or twice daily, the exact time being noted in each case. When the hydrometer reading has decreased to a value of about 1·008, the short-stem hydrometer should be used. The long-stem hydrometer should be used for two further parallel readings as a check and then put aside.

3·91 Throughout the test the temperature should be controlled as far as possible. To avoid unsymmetrical heating and consequent convection currents, the suspension should be kept out of direct sunlight and away from any local source of heat. Some form of lid on the measuring cylinder is useful for retarding evaporation. The temperature of the suspension should be checked for each reading and the average temperature over a period of the test should not differ from the mean temperature by more than 2°C., in order not to cause an error in the particle size of more than 2 per cent. This requirement is

(A) MOISTURE CONTENT TINS AND BOTTLE

(B) PYCNOMETER

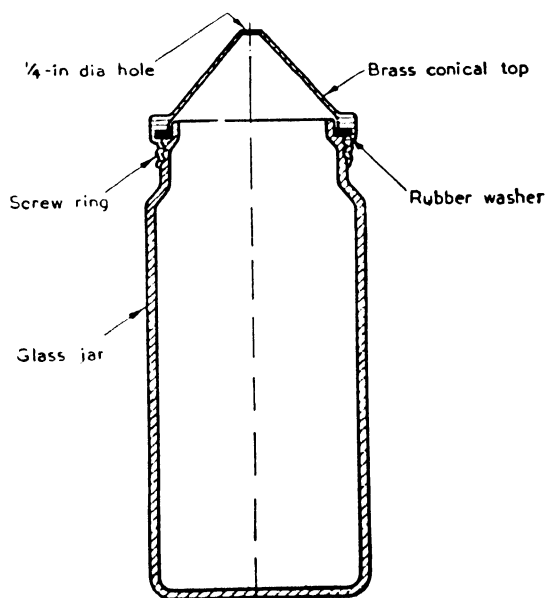
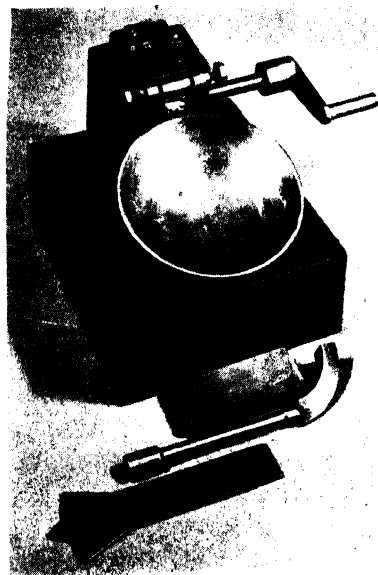
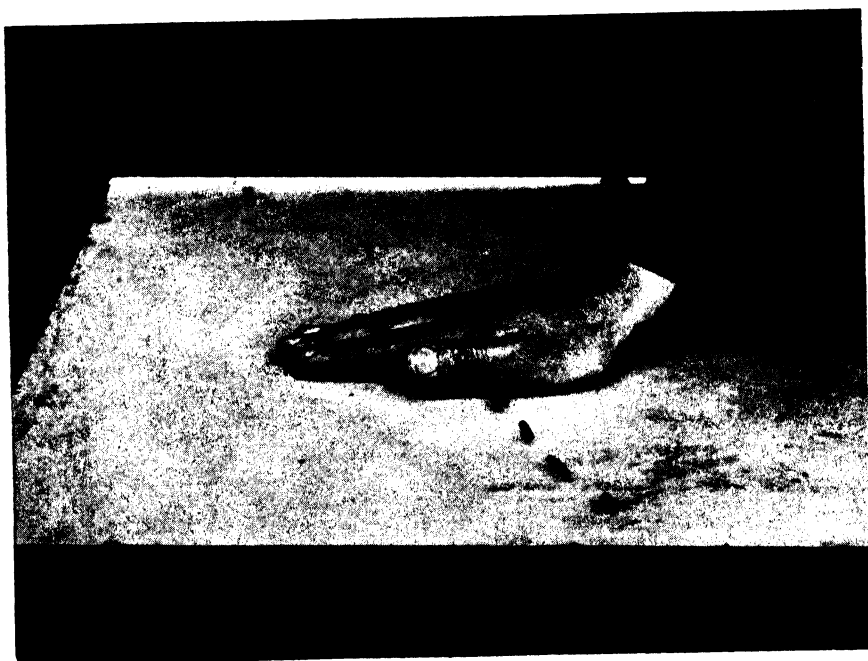
PLATE 3·1

(A) LIQUID LIMIT DEVICE AND TOOLS

(B) DETERMINING THE PLASTIC LIMIT OF SOIL

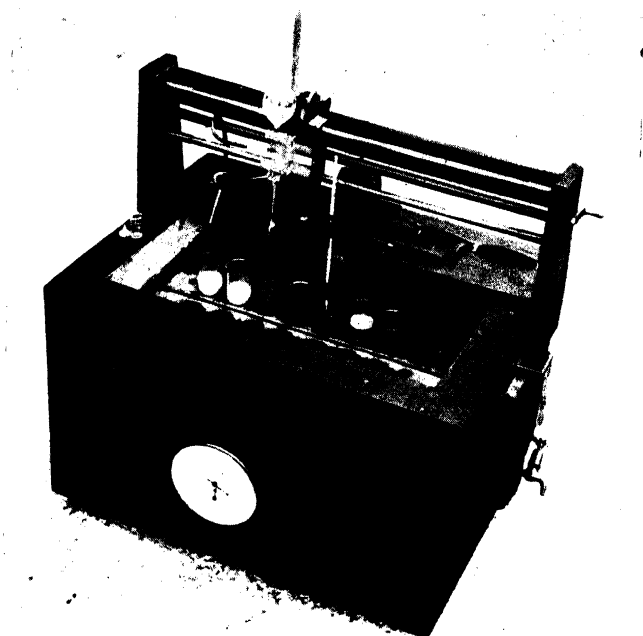
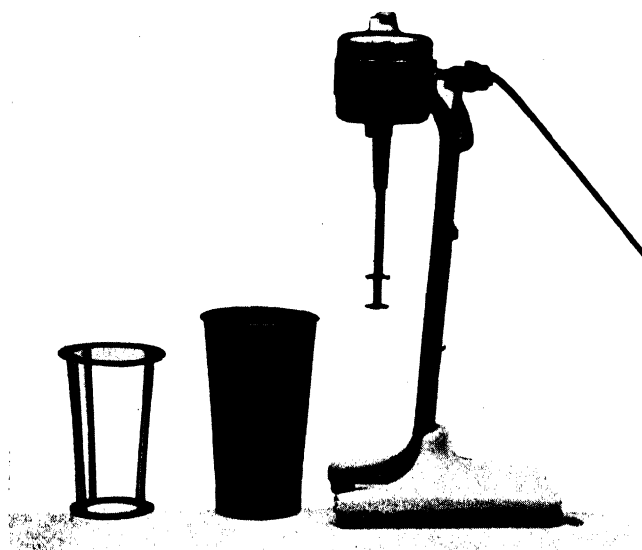
PLATE 3·2

(A) CONSTANT-TEMPERATURE BATH

(B) HIGH-SPEED MECHANICAL STIRRER
with cup and wire baffle

PLATE 3·3

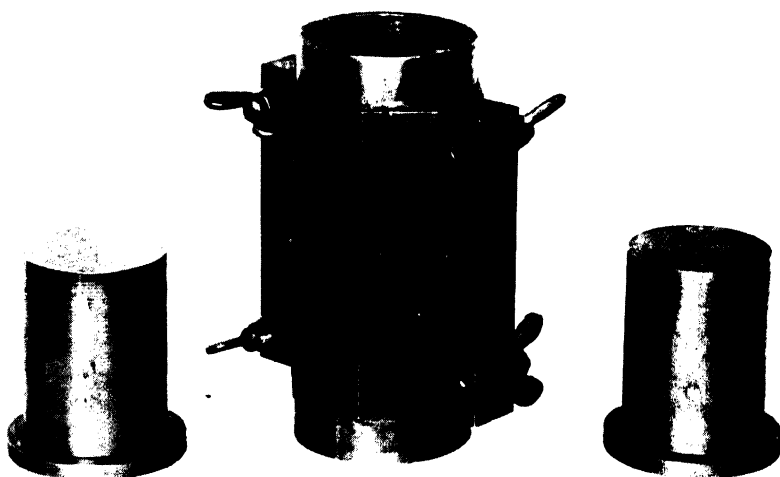
(A) BÜCHNER FUNNEL
for filtering soil solution

(B) CONSTANT-VOLUME MOULD
for making specimens 2 in. in diameter by ½ in. high for shrinkage limit determinations

PLATE 3·4

generally fulfilled if the maximum difference in room temperature is not greater than 8°C.

3·92 CALCULATIONS. The observed data and computed quantities are assembled for convenience in a table containing the following columns:—

I	II	III	IV	V	VI	VII	VIII	IX
Date	Time	Temp.	Elapsed time	R'_h	$R_h = R'_h + C_m$	D (mm.)	$R_h + m_t - c_d$	W (%)

where R'_h = the hydrometer reading at the upper rim of the meniscus. In recording the reading the units figures are ignored and the decimal point placed between the third and fourth decimal places, e.g. the density 1·0325 should be read as 32·5.

C_m = the meniscus correction.

c_d = 0·8, which is the correction for the presence of sodium oxalate in the suspension.

m_t = the temperature correction obtained from the temperature correction chart shown in the appendix to this chapter.

3·93 The equivalent particle diameter (D) corresponding to the elapsed time (t) at each hydrometer reading can be obtained by means of the nomographic chart from the experimental data. Thus, a value of the constant " B " is first obtained by placing a straight-edge across the specific gravity (G_s) and temperature scales at the appropriate values. A value for the particle velocity (v) is also obtained by placing a straight-edge across the hydrometer reading (R_h) and time (t) scales at the experimental values. Finally, a straight-edge is placed across the " B " and velocity scales at the value determined above, and the equivalent particle diameter (D) then read from the appropriate scale. This figure should be recorded in column VII of the table.

3·94 After the corrected hydrometer reading has been obtained by means of the temperature and dispersing agent corrections (m_t and c_d) it should be recorded in column VIII. The percentage by weight (W) of particles smaller than the corresponding equivalent particle diameters is then found from the formula:—

$$W = \frac{100\ G_s}{W_s\ (G_s - 1)}\ (R_h + m_t - 0{\cdot}8)\ \text{(per cent)}$$

where W_s = total dry weight of soil particles in 1,000 ml. of suspension.

G_s = specific gravity of soil particles.

3·95 The values of W are calculated for each value of D and the results entered in column IX of the table. The results of the sieve analysis of the gravel and the sand fractions are obtained in the same way as in the standard laboratory method and expressed as cumulative percentages of the total sample. They can then be plotted as a particle-size distribution curve as before, from which the proportions of material within the various size fractions (sand, silt and clay) may be obtained and reported in tabular form.

DETERMINATION OF THE SHRINKAGE LIMIT OF SOIL

Definition

3·96 When a wet clay containing no air voids loses water the total volume decreases; the decrease is equal initially to the volume of water lost. However, at a certain point during the drying process air begins to enter the soil and the volume decrease becomes appreciably less than the volume of water lost until, when the soil is very dry, changes in moisture content cause only slight volume changes in the soil structure. Although in practice there is often only a gradual transition between the two stages, a theoretical limit can be deduced graphically from the volume/moisture content relationship, below which appreciable shrinkage does not occur. This limiting moisture content is known as the shrinkage limit (SL) of the soil.

Uses of the Test

3·97 The shrinkage limit has been used in soil classification, particularly in the U.S. Public Roads Administration method[6], although in the revised Public Roads system this test has been dispensed with [7]. Experience at the Road Research Laboratory has indicated that the shrinkage limit of soils in the British Isles is relatively constant at about 12 to 14 per cent, but values differing from these may be obtained with soils from other areas.

3·98 The shrinkage limit considered in relation to the natural moisture content of soil in the field indicates whether or not further shrinkage will occur if the soil is allowed to dry out. In this country the moisture content of clays is always well above the shrinkage limit except in surface strata which are subject to air-drying.

Review of Methods

3·99 Most of the methods used for finding the shrinkage limit of soil involve the measurement of the total volume of a specimen as it dries out. This is usually done by determining the displacement when the specimen is immersed in mercury[2] [8] or by direct measurement. An apparatus of the mercury displacement type has been developed at the Road Research Laboratory. This apparatus has the advantage that it may be used to determine the shrinkage of undisturbed samples taken with a 2-in. diameter sampling tube.

Method of Test

3·100 APPARATUS. The apparatus required is:—

(1) The mercury displacement apparatus shown in Fig. 3·6.

(2) A constant-volume mould of the type shown in Plate 3·4B.

(3) A balance capable of weighing up to 200 gm accurate to 0·01 gm.

3·101 PROCEDURE. The displacement apparatus consists of a metal vessel containing mercury in which the specimen, supported in a small metal cage, can be totally immersed. In operation the empty cage is completely immersed in the mercury and the dial gauge (G) set to zero by means of the screw adjustment (S); the attached contact (A) is then adjusted until an electrical circuit is just made with the mercury, as indicated by the lighting of a small bulb.

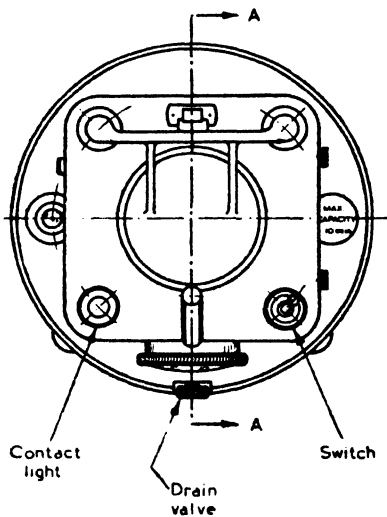
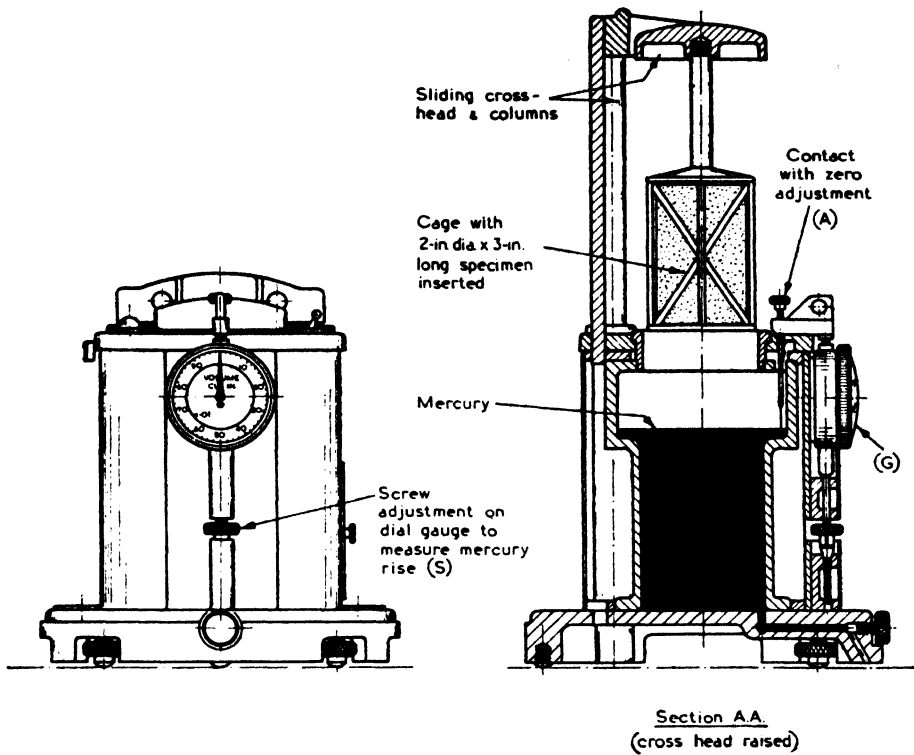



FIG. 3·6 SOIL VOLUME MEASURING APPARATUS
for 2-in. diameter specimens

The cage is raised, a specimen placed in it and the whole re-immersed. The contact point is raised until the surface of the mercury is again reached, as indicated by the bulb, the zero adjustment remaining untouched. The rise of the mercury is then read directly on the dial gauge.

3·102 To calibrate the apparatus a small steel disc 2 in. in diameter and 0·5 in. high is immersed in the mercury, and the level is noted before and after immersion. The rise in level (h_1), is noted and the cross-sectional area (A) of the immersion vessel is then given by the formula:—

$$A = \frac{25 \cdot 74}{h_1} \quad (\text{ml.})$$

3·103 Soil specimens are then made up in the form of discs 2 in. in diameter and 0·5 in. thick, in the constant-volume mould. Alternatively, specimens 0·5 in. thick may be cut from undisturbed samples 2 in. in diameter taken in the field by means of a sampling tool.

3·104 When disturbed soil is used, dry material is first mixed with sufficient water to bring it to the plastic limit (PL) of the soil, and sufficient of the wet soil is placed in the mould to fill it completely when the two end pieces are pressed home. The weight of wet soil (W) can be calculated from the formula:—

$$W = \frac{25 \cdot 74\ G_S\ (100 + PL)}{100 + G_S\ PL}$$

where G_S is the specific gravity of the soil. The compacted specimen is then removed from the mould and the volume determined in the manner described. If h_2 is the rise in level of the mercury, the volume is given by the formula:—

$$V_1 = Ah_2 \quad (\text{ml.})$$

where A is the cross-sectional area of the immersion vessel determined in the calibration experiment. The specimen is weighed accurately to the nearest 0·01 gm (W_1) and both the weight and volume recorded.

3·105 Determinations of volume and weight are repeated at frequent intervals while the specimen is drying out. The drying may initially be done by allowing the specimen to stand in the open air in the laboratory, or more rapidly, in a desiccator over calcium chloride for various periods. During the final stages of drying it may be convenient to stand the specimens on the top of an oven used for determinations of moisture content.

3·106 Finally, the specimen is dried out completely in an oven at 105° to 110°C. and the weight of dry soil in it is determined (W_S).

3·107 CALCULATIONS. The volumes occupied by the specimen at different moisture contents are, for convenience, expressed in terms of the corresponding volumes occupied by a sample containing 100 gm of dry soil. If the volume of the sample containing a weight of dry soil (W_S) is V_1, the corresponding volume (V_2) of a sample containing 100 gm of dry soil is given by:—

$$V_2 = \frac{100}{W_S} V_1 \quad (\text{ml.})$$

3·108 The corresponding values of the moisture contents are calculated from the formula:—

$$m = \frac{100\,(W_1 - W_S)}{W_S} \quad \text{(per cent)}$$

and the two quantities are plotted on a graph in which the values of volume are shown on the y axis. The type of curve obtained is shown in Fig. 3·7. The straight part of the curve so obtained is extrapolated to cut the abscissa corresponding to the volume occupied by the completely dried soil. The moisture content corresponding to this point of intersection is read off and recorded as the shrinkage limit (SL) of the soil. It is usually advisable to carry out duplicate determinations, and record the average value as the shrinkage limit. The shrinkage limit is usually reported to the nearest 1 per cent.

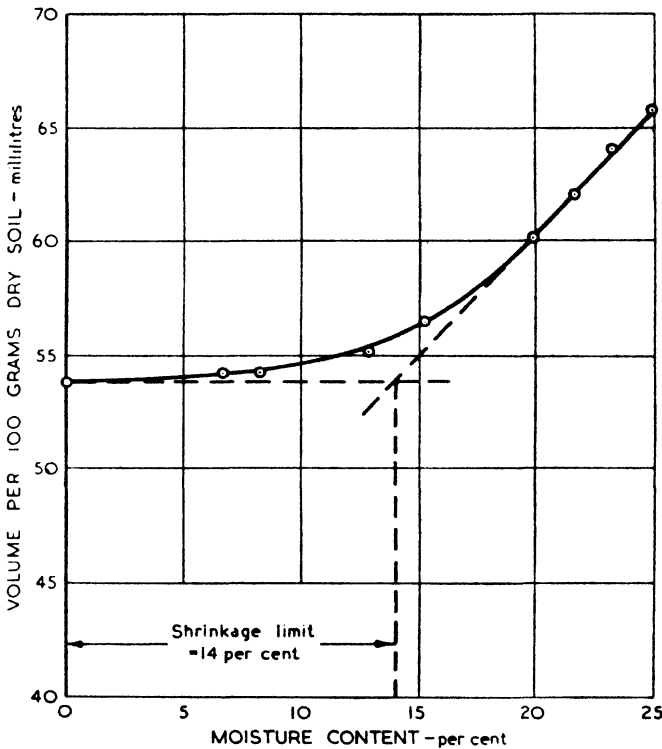



FIG. 3·7 SHRINKAGE LIMIT TEST FOR SOIL
Relationship between the volume of soil and its moisture content.

THE FIELD MOISTURE EQUIVALENT AND CENTRIFUGE MOISTURE EQUIVALENT OF SOIL

Determinations of the Field Moisture Equivalent (F.M.E.) and Centrifuge Moisture Equivalent (C.M.E.)[2] [9]

3·109 These were originally included as tests in the U.S. Public Roads

Administration classification, although they are now omitted. They are arbitrary tests introduced by early agricultural research workers in an attempt to compare the ability of different soils to hold water. They have little physical significance and are not now widely used.

3·110 THE FIELD MOISTURE EQUIVALENT. This is the maximum moisture content at which a drop of water placed on the smooth surface of the soil will not be absorbed immediately but will spread out over the surface and give it a shiny appearance.

3·111 The test is performed by mixing water with the soil fraction that passes a No. 36 B.S. sieve until the soil forms balls. Small increments are then made until a drop of water fails to penetrate the smoothed surface of the soil pat, This test does not give reproducible results.

3·112 THE CENTRIFUGE MOISTURE EQUIVALENT. This is the moisture content retained by a soil that has first been saturated with water and then subjected to a force equal to 1,000 times the force of gravity for 1 hour.

3·113 The test consists of first soaking a small sample of air-dried soil with water in a Gooch crucible, then draining it in a humidifier for at least 12 hours and, finally, centrifuging it for 1 hour.

3·114 The C.M.E. is intended to measure the capacity of a soil to hold water. Its value appears to increase linearly with the clay content of a soil.

3·115 Since the centrifuging process is carried out for one hour only, equilibrium moisture conditions may not be achieved in the specimen. The C.M.E. does not therefore necessarily represent a point on the soil suction/moisture content relationship (see Chapter 16).

APPENDIX

Data Sheets and Worked Examples

3·116 Comprising:—

(1) Determination of liquid limit, plastic limit and natural moisture content of soil (Data Sheet 3·A).

(2) Determination of specific gravity of soil particles (Data Sheet 3·B).

(3) Determination of particle-size distribution. Pretreatment of soil (Data Sheet 3·C).

(4) Determination of particle-size distribution of soil by pipette method.
 (i) Fine analysis (Date Sheet 3·D).
 (ii) Coarse analysis (Data Sheet 3·E).
 (iii) Particle-size distribution curve (Chart 3·A).

(5) Determination of particle-size distribution of soil by hydrometer method.
 (i) Sedimentation analysis (Data Sheet 3·F).
 (ii) Solution of Stokes' Law (Charts 3·B and 3·C).
 (iii) Total analysis (Data Sheet 3·G).
 (iv) Particle-size distribution curve (Chart 3·D).

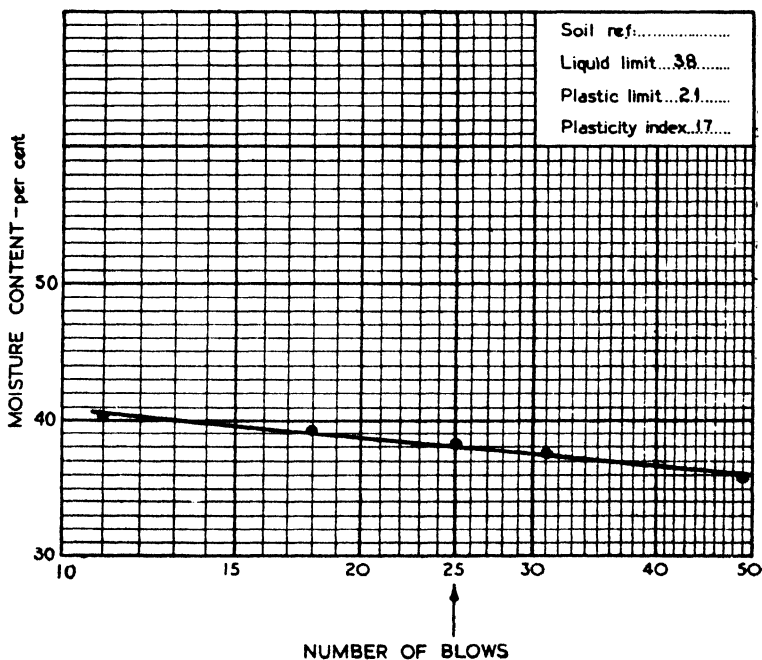
DATA SHEET 3·A

DETERMINATION OF THE NATURAL AND AIR-DRY MOISTURE CONTENT AND THE LIQUID AND PLASTIC LIMITS OF A SOIL

OPERATOR:
DATE:
Moisture content test to be marked m
Liquid Limit test to be marked LL
Plastic Limit test to be marked PL
Air-dry moisture content to be marked A.

JOB:
SHEET No.
No. of blows refers to liquid limit determination. Moisture content

$$= m = \frac{\text{Wt. of moisture}}{\text{Wt. of dry soil}} \times 100\ (\%)$$

Sample No.	1	2	3	4	5	6	7	8	9	10
Type of test	*LL*	*LL*	*LL*	*LL*	*LL*		*PL*	*PL*		*m*
No. of blows	*49*	*31*	*25*	*18*	*11*					
Container No.	*264*	*233*	*144*	*289*	*49*		*192*	*1*		*19*
Wt. of wet soil + container (gm)	*54.67*	*52.18*	*55.19*	*53.98*	*52.63*		*37.75*	*37.69*		*51.92*
Wt. of dried soil + container (gm)	*48.15*	*46.12*	*48.23*	*47.23*	*46.13*		*36.37*	*36.37*		*46.41*
Wt. of container (gm)	*30.00*	*30.00*	*30.00*	*30.00*	*30.00*		*30.00*	*30.00*		*30.00*
Wt. of moisture (gm)	*6·52*	*6.06*	*6.96*	*6.75*	*6.50*		*1.38*	*1.32*		*5.51*
Wt. of dry soil (gm)	*18.15*	*16.12*	*18.23*	*17.23*	*16.13*		*6.37*	*6.37*		*16.41*
Moisture content (m) (%)	*35.9*	*37.6*	*38.2*	*39.2*	*40.3*		*21.7*	*20.7*		*33.6*

DATA SHEET 3·B

DETERMINATION OF THE SPECIFIC GRAVITY OF SOIL PARTICLES (LABORATORY AND FIELD METHODS)

OPERATOR: JOB:
DATE: SAMPLE No:
METHOD: LABORATORY* FIELD TEMPERATURE OF TEST

Test No.		1	2	3	4	5
Wt. of bottle + soil + water (W_3)	(gm)	*83.025*	*84.977*			
Wt. of bottle + soil (W_2)	(gm)	*38.259*	*40.701*			
Wt. of bottle (W_1)	(gm)	*24.144*	*25.123*			
Wt. of water used ($W_3 - W_2$)	(gm)	*44.766*	*44.276*			
Wt. of soil used ($W_2 - W_1$)	(gm)	*14.115*	*15.578*			
Vol. of soil ($50 - (W_3 - W_2)$)	(ml.)	*5.203*	*5.724*			
Specific gravity of soil $G_s = \frac{W_2 - W_1}{50 - (W_3 - W_2)}$		*2.70*	*2.72*			

*Strike out the word that does not apply.

DATA SHEET 3·C

DETERMINATION OF PARTICLE-SIZE DISTRIBUTION IN SOILS

OPERATOR: JOB:
DATE: SAMPLE No:

PRETREATMENT OF SOIL

Test No.		1	2	......	10
Wt. of soil + evaporating dish before pretreatment	(gm)	*323*	*318*		
Wt. of soil + evaporating dish after pretreatment	(gm)	*317*	*310*		
Wt. of evaporating dish	(gm)	*220*	*211*		
Original wt. of unpretreated soil	(gm)	*103*	*107*		
Loss in weight after pretreatment	(gm)	*6*	*8*		
Loss in weight after pretreatment	(%)	*6*	*7*		

Test No.				......	
Wt. of soil + evaporating dish before pretreatment	(gm)				
Wt. of soil + evaporating dish after pretreatment	(gm)				
Wt. of evaporating dish	(gm)				
Original wt. of unpretreated soil	(gm)				
Loss in weight after pretreatment	(gm)				
Loss in weight after pretreatment	(%)				

DATA SHEET 3·D

DETERMINATION OF PARTICLE-SIZE DISTRIBUTION OF A SOIL (BRITISH STANDARD LABORATORY METHOD)

FINE ANALYSIS

OPERATOR: **JOB:**

DATE: **WEIGHT OF SOIL SAMPLE:** *10.000 gm.*

Temperature of Sedimentation Bath: *25° C.* **Volume of Pipette:** *10.15 ml.*
Time of taking 1st pipette sample: *4* **min.** *8* **sec.**
Time of taking 2nd pipette sample: *46* **min.** *0* **sec.**
Time of taking 3rd pipette sample: *6* **hr** *54* **min.**

Soil Specimen No.	*A2*		
Weighing bottle No.	*1*		
Wt. of bottle and soil retained on 25 B.S. sieve	*21.746*		
Wt. of bottle	*18.084*		
Wt. of soil retained on 25 B.S. sieve	*3.662*		
Percentage material 2.0 mm. — 0.6 mm.	*36.62*		
Weighing bottle No.	*2*		
Wt. of bottle and soil retained on 72 B.S. sieve	*20.942*		
Wt. of bottle	*18.084*		
Wt. of soil retained on 72 B.S. sieve	*2.858*		
Percentage material 0.6 mm. — 0.2 mm.	*28.58*		
Weighing bottle No.	*3*		
Wt. of bottle and soil retained on 200 B.S. sieve	*18.654*		
Wt. of bottle	*18.084*		
Wt. of soil retained on 200 B.S. sieve	*0.570*		
Percentage material 0.2 mm. — 0.06 mm.	*5.70*		
Weighing bottle No.	*114*		
Wt. of bottle and 1st pipette sample	*16.835*		
Wt. of bottle	*17.797*		
Wt. of 1st pipette sample (10 ml. susp.)	*0.038*		
Equivalent wt. in 500 ml. suspension	*1.872*		
Corrected wt. in 500 ml. suspension	*1.472*		
Percentage material < 0.02 mm.	*14.72*		
Weighing bottle No.	*22*		
Wt. of bottle and 2nd pipette sample	*14.653*		
Wt. of bottle	*14.627*		
Wt. of 2nd pipette sample (10 ml. susp.)	*0.026*		
Equivalent wt. in 500 ml. suspension	*1.281*		
Corrected wt. in 500 ml. suspension	*0.881*		
Percentage material < 0.006 mm.	*8.81*		
Weighing bottle No.	*75*		
Wt. of bottle and 3rd pipette sample	*17.386*		
Wt. of bottle	*17.364*		
Wt. of 3rd pipette sample (10 ml. susp.)	*0.022*		
Equivalent wt. in 500 ml. suspension	*1.084*		
Corrected wt. in 500 ml. suspension	*0.684*		
Percentage material < 0.002 mm.	*6.84*		
Coarse Sand 2.0 mm. — 0.6 mm. (%)	*37*		
Medium Sand 0.6 mm. — 0.2 mm. (%)	*29*		
Fine Sand 0.2 mm. — 0.06 mm. (%)	*6*		
Coarse Silt 0.06 mm. — 0.02 mm. (%)	*13*		
Medium Silt 0.02 mm. — 0.006 mm. (%)	*6*		
Fine Silt 0.06 mm. — 0.002 mm. (%)	*2*		
Clay < 0.002 mm. (%)	*7*		
	100	100	100

DATA SHEET 3·E

DETERMINATION OF PARTICLE-SIZE DISTRIBUTION OF A SOIL (STANDARD LABORATORY METHOD)

(For use when material retained on the No. 7 B.S. sieve is present)

OPERATOR: JOB:

DATE: SAMPLE No. *A2*

Total weight of sample = *4400* gm

Weight retained on No. 7 B.S. sieve = *2200* gm

Coarse Analysis

B.S. Sieve	Wt. retained (gm)	% retained	Cumulative % passing	Cumulative passing %
$1\frac{1}{2}$ in.	*0*	*0*	*100*	*100*
$\frac{3}{4}$ in	*396*	*9*	*91*	*91*
$\frac{3}{8}$ in.	*704*	*16*	*75*	*75*
$\frac{3}{16}$ in.	*660*	*15*	*60*	*60*
No. 7	*440*	*10*	*50*	*50*
< No. 7	*2200*	*50*		
Total	*4400*	*100*		

Fine Analysis (from Data Sheet 3·D)

Fraction	%	Size (mm.)	Cumulative % passing	Cumulative passing %
Coarse Sand (2.0 mm. — 0.6 mm.)	*37*	2	*100*	*50*
Medium Sand (0.6 mm. — 0.2 mm.)	*29*	0.6	*63*	*32*
Fine Sand (0.2 mm. — 0.06 mm.)	*6*	0.2	*34*	*17*
Coarse Silt (0.06 mm. — 0.02 mm.)	*13*	0.06	*28*	*14*
Medium Silt (0.02 mm. — 0.006 mm.)	*6*	0.02	*15*	*7*
Fine Silt (0.006 mm. — 0.002 mm.)	*2*	0.006	*9*	*5*
Clay (< 0.002 mm.)	*7*	0.002	*7*	*3*
Total	*100*			

Sand (2.0 mm. — 0.06 mm.)	*72*
Silt (0.06 mm. — 0.002 mm.)	*21*
Clay (< 0.002 mm.)	*7*
Total	*100*

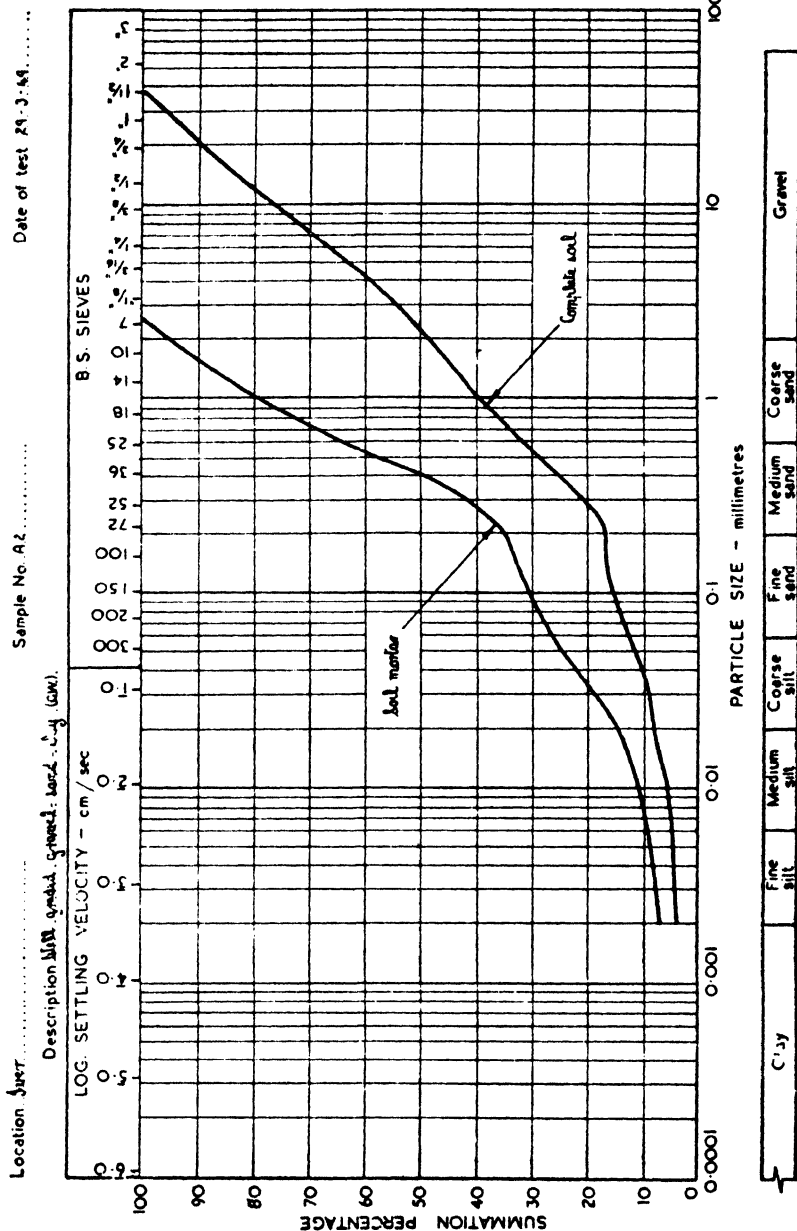



CHART 3·A. CHART FOR RECORDING PARTICLE-SIZE DISTRIBUTION

DATA SHEET 3·F

DETERMINATION OF PARTICLE-SIZE DISTRIBUTION OF A SOIL (SUBSIDIARY FIELD METHOD)

FINE ANALYSIS

OPERATOR: **JOB:**

DATE: **SAMPLE NO:** *C2*

Hydrometer No: *2* **S.G. of soil particles (G_s):** *2.65*

Meniscus correction (C_m) : *0.5*

$$W\% = \frac{100\, G_s}{W_s\,(G_s - 1)} \times (R_h + m_t - 0.8) = \frac{100}{40} \times \frac{2.65}{1.65} \times (R_h + m_t - 0.8)$$

Date	Time	Temp.	Elapsed time	Hydro-meter reading (R'_h)	Corrected hydrometer reading $R_h = R'_h + C_m$	Equivalent particle diameter D (mm.)	$R_h + m_t - 0.8$	Percentage of particles finer than the corresponding particle diameter
	10.12	*21.9*	*30 sec.*	*21.5*	*22.0*	*0.064*	*21.4*	*85.9*
			1 min.	*21.0*	*21.5*	*0.045*	*20.9*	*83.9*
			2 min.	*20.3*	*20.8*	*0.032*	*20.2*	*81.1*
			4 min.	*19.3*	*19.8*	*0.023*	*19.2*	*77.1*
		21.7	*8 min.*	*18.9*	*19.4*	*0.016*	*18.8*	*75.5*
		21.5	*15 min.*	*17.9*	*18.4*	*0.012*	*17.8*	*71.5*
		21.1	*30 min.*	*16.7*	*17.2*	*0.009*	*16.6*	*66.7*
		21.0	*1 hr 2 min.*	*16.0*	*16.5*	*0.0062*	*15.9*	*63.8*
		20.6	*2 hr*	*15.0*	*15.5*	*0.0045*	*14.9*	*59.8*
		20.5	*4 hr 4 min.*	*14.0*	*14.5*	*0.0032*	*13.9*	*55.8*
		22.4	*24hr 19 min*	*11.9*	*12.4*	*0.0013*	*11.8*	*47.4*

Evaporating dish No.			
Wt. of dish and dried soil		*231.4*	gm
Wt. of dish		*191.4*	gm
Wt. of dry soil	(W_s)	*40.0*	gm

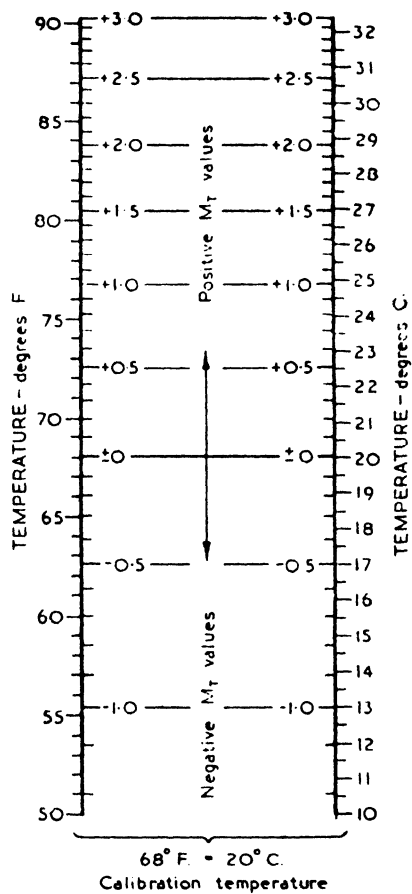



CHART 3·B TEMPERATURE CORRECTION CHART
For hydrometers calibrated in density at 20°C.

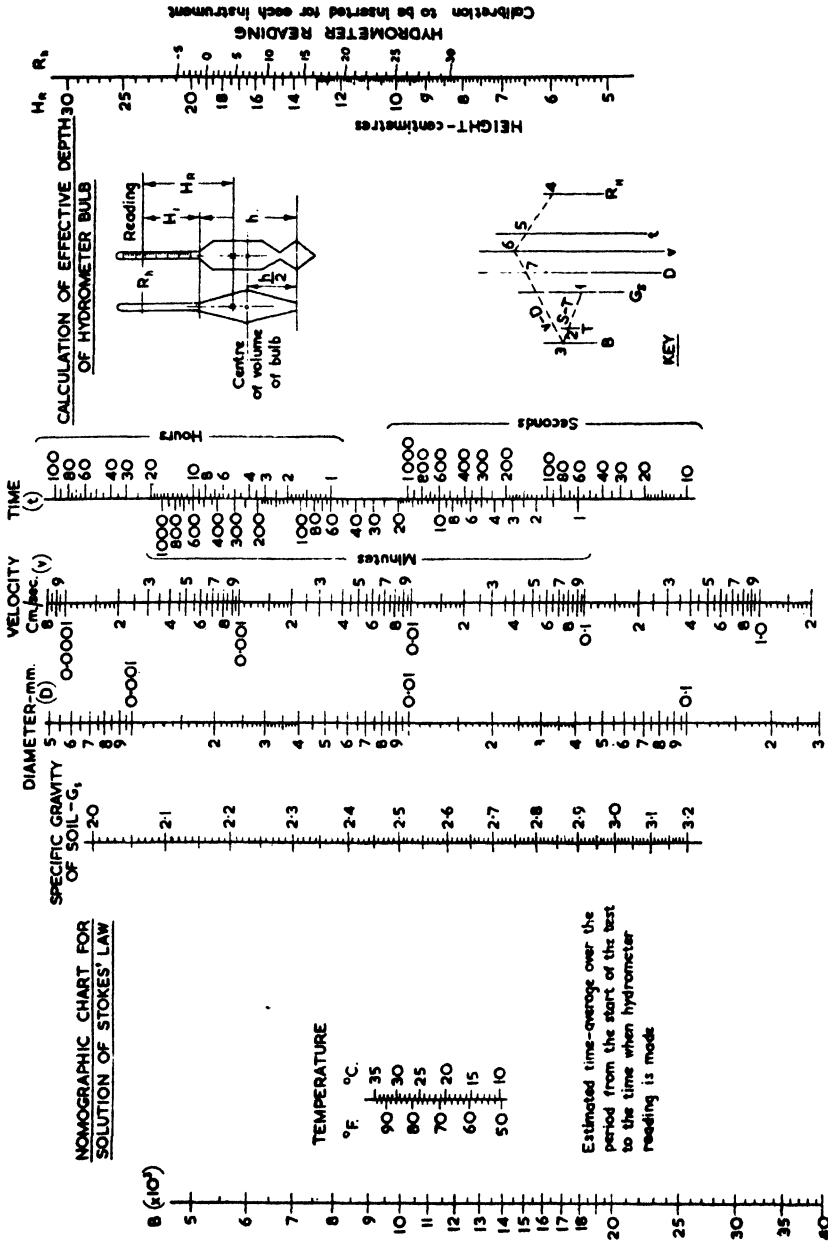



CHART 3·C NOMOGRAPHIC CHART FOR USE WITH HYDROMETER

DATA SHEET 3·G

DETERMINATION OF PARTICLE-SIZE DISTRIBUTION OF A SOIL (SUBSIDIARY FIELD METHOD)

OPERATOR: **JOB:**
DATE: **SAMPLE No:** *C2*

Total wt. of sample = *40* gm
Weight retained on No. B.S. 7 sieve = *Nil* gm

Coarse Sieve Analysis

B.S. Sieve	Wt. retained (gm)	% retained	Cumulative % passing	Cumulative % passing
1½ in. ¾ in. ⅜ in. 3/16 in. No. 7 <No. 7	*Nil*			
Total				
Fine Sieve Analysis				
No. 7	*0*	*0*	*100*	*100*
No. 14	*0.44*	*1.1*	*98.9*	*99*
No. 25	*0.68*	*1.7*	*97.2*	*97*
No. 52	*0.92*	*2.3*	*94.9*	*95*
No. 100	*0.96*	*2.4*	*92.5*	*93*
No. 200	*0.40*	*1.0*	*91.5*	*91*
<No. 200	*36.60*			
Total	*40.00*			

Sedimentation Analysis

Equivalent particle diameter (mm.)	Cumulative % passing	Cumulative % passing
0.064	*85.9*	*86*
0.045	*83.9*	*84*
0.032	*81.1*	*81*
0.023	*77.1*	*77*
0.016	*75.5*	*76*
0.012	*71.5*	*71*
0.009	*66.7*	*67*
0.0062	*63.9*	*64*
0.0045	*59.8*	*60*
0.0032	*55.8*	*56*
0.0013	*47.4*	*47*

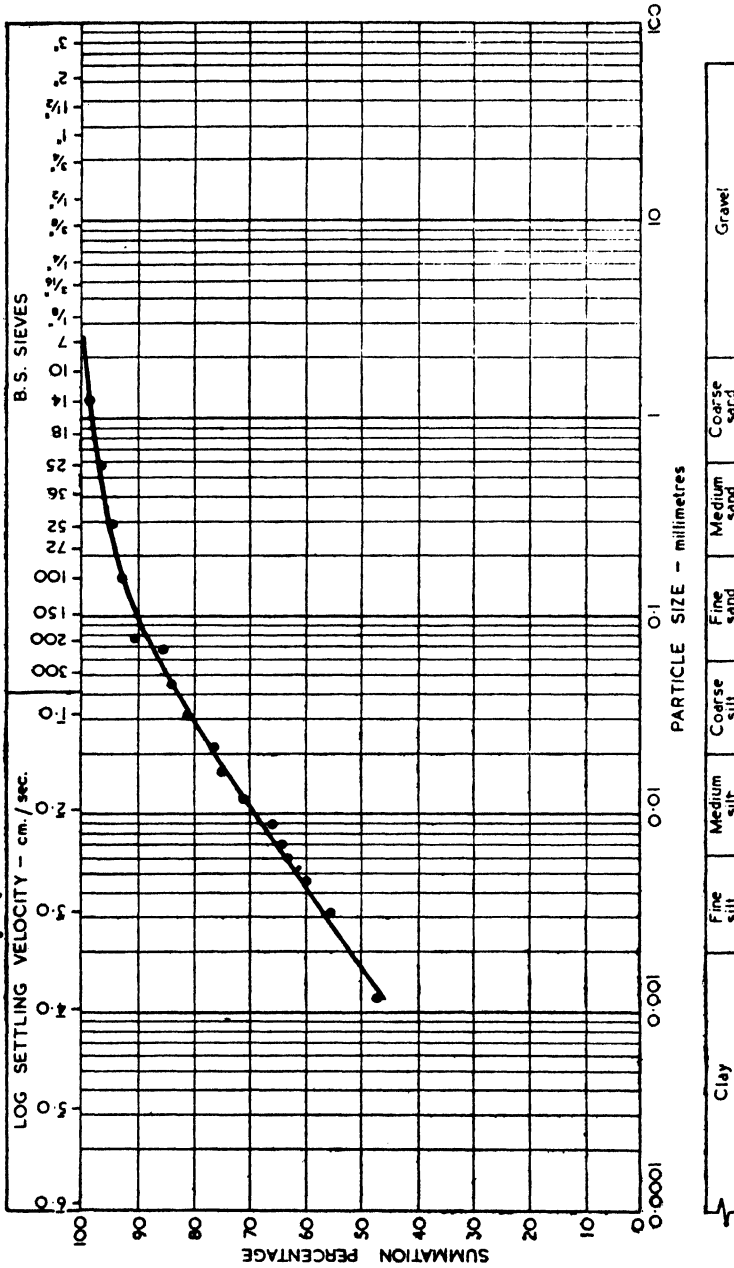



CHART 3·D. CHART FOR RECORDING PARTICLE-SIZE DISTRIBUTION
(showing results of hydrometer analysis)

SUMMARY

3·117 This chapter describes the tests for identifying and classifying soils for engineering purposes. The procedures, which are given in detail, comprise methods for the determination of the moisture content of soil and the density of soil particles, as well as for evaluating the shrinkage and plasticity characteristics of cohesive soils. Methods are also given for determining the particle-size distribution in soils. In most cases the account of a standard laboratory method is supplemented by a description of a method that will give equivalent results under field conditions. Worked data sheets are also included, showing the results obtained in typical tests.

REFERENCES TO CHAPTER 3

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CHAPTER 4

THE CLASSIFICATION OF SOILS

INTRODUCTION

4·1 This chapter describes in detail the principal systems of classifying soils for civil engineering purposes. The use of these different systems has led to much confusion in the technical literature relating to soils and their properties. This has been so marked that it has been thought worth while to collect them together and to refer particularly to the contributions of Casagrande.

4·2 The original classification devised by Casagrande was not entirely satisfactory for certain groups of soils. He subsequently suggested an extension of his classification, and his original classification together with those parts of the extension which apply to British soils have been adopted for use at the Road Research Laboratory.

4·3 The Code of Practice for Site Investigations[1] recommends the use of this extended form of the Casagrande system for classifying soils for roads and airfields.

4·4 The classification of a soil according to any of the accepted systems does not in itself enable detailed studies of soils to be dispensed with altogether. For instance, while it is known that a predominantly sandy soil usually forms a good subgrade for roads, its stability in any particular circumstances will depend on the moisture content and compaction of the soil and these are not indicated by the classification.

CASAGRANDE CLASSIFICATION SYSTEM

4·5 The Casagrande classification system[2] was originally devised in 1942 when it was adopted by the U.S. Corps of Engineers for use in connexion with airfield construction.

4·6 Casagrande has suggested an extension to his classification which included, amongst others, two additional sub-groups. Since these were particularly suitable for classifying British soils, they have been adopted for use at the Road Research Laboratory.

4·7 The main soil groups of the extended classification are given in Table 4·1. As shown in column 4 of the chart, soils are designated by group symbols consisting of a prefix and a suffix. The prefixes indicate the six main soil types—gravel, sand, silt, clay, fine-grained organic soil and peat—while the suffixes indicate subdivisions of these groups. Each coarse-grained soil type is subdivided into five sub-groups and each fine-grained soil type into three sub-groups.

4·8 The first six columns of Table 4·1 indicate how soils from each group are identified and classified both in the laboratory and in the field. The remaining columns indicate general characteristics of the various soil types.

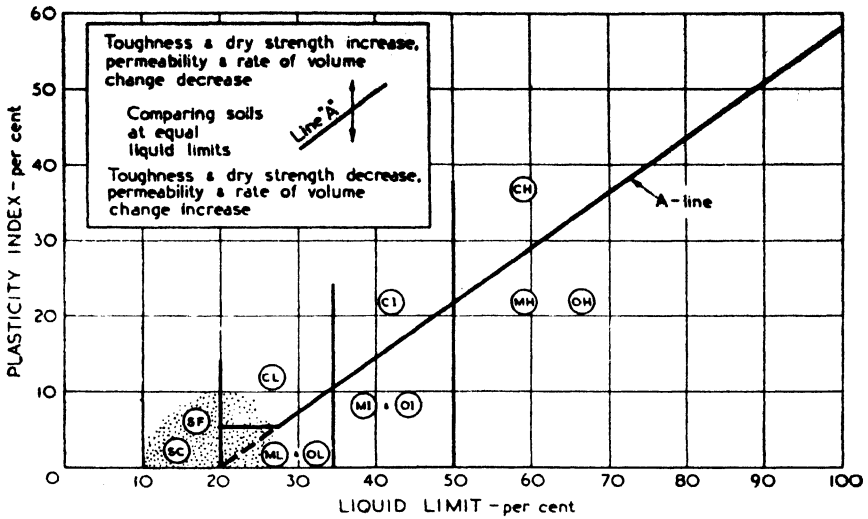



FIG. 4·1 PLASTICITY CHART USED IN CASAGRANDE SOIL CLASSIFICATION

(Extended to include intermediate groups)

4·9 A plasticity chart is used in conjunction with Table 4·1 when classifying fine-grained soils. This chart is shown in Fig 4·1.

4·10 The extended system as used at the Road Research Laboratory takes the following form of grouping:—

	MAIN SOIL TYPE	PREFIX
Coarse-grained soils	gravel	G
	sand	S
Fine-grained soils	silt	M
	clay	C
	organic silts and clays	O
Fibrous soils	peat	Pt

4 11 The suffixes indicate the following subdivisions:—

	SUBDIVISION	SUFFIX
For coarse-grained soils	well graded, with little or no fines	W
	well graded, with suitable clay binder	C
	uniformly graded, with little or no fines	U
	poorly graded, with little or no fines	P
	poorly graded with appreciable fines, or well graded with excess fines.	F
For fine-grained soils	low compressibility (LL<35)	L
	medium compressibility (LL = 35-50)	I
	high compressibility (LL>50)	H

4·12 In the above scheme the terms used in describing the particle-size distribution of the coarse-grained soils have these meanings:—

Well graded —having a particle-size distribution extending evenly over a wide range of particle sizes, without excess or deficiency of any particular sizes.

Poorly graded —having a particle-size distribution containing an excess of some particle sizes and a deficiency of others.

Uniformly graded —having a particle-size distribution extending over a very limited range of particle sizes, i.e. poorly graded but with an excess of only one small range of particle sizes and with a deficiency of all others.

(Closely graded —has the same meaning as " uniformly graded.")

4·13 The coarse-grained soils are classified mainly on the basis of their particle-size analysis; the fine-grained soils are classified on the basis of their plasticity characteristics with the aid of the plasticity chart shown in Fig. 4·1; the fibrous soils are readily identified by visual examination.

4·14 The suffix U which has been introduced into the extended classification includes many gravels, clean ballasts, beach and dune sands which would have been designated by the suffix P in the original system.

4·15 Similarly the suffixes L and H in the original system tended to be inadequate in covering the whole range of plasticity, particularly in this country where there is such a predominance of clays that a further subdivision would be useful. Casagrande and others[2] [3] [4] suggested the inclusion of the intermediate group I to cover part of the L range previously used.

Identification of Coarse-grained Soils

4·16 The identification of coarse-grained soils is based chiefly upon the particle-size distribution. Although the predominant particle-size is usually recognized quite easily[5], a particle-size analysis helps to distinguish between well graded and poorly graded soils.

4·17 The GW and SW soils are well graded gravelly and sandy soils which have little or no fines. Less than 5 to 10 per cent should pass the No. 200 B.S. sieve.

4·18 The GC and SC soils are also well graded but contain sufficient binder to provide cohesion without undue swelling or shrinkage. Although the binder is usually clay, these groups also include binders such as calcareous materials or iron oxides. In order to identify a clay binder the material passing a No. 36 B.S. sieve should be examined for plasticity and dry strength in the same way as for a fine-grained soil.

4·19 The GP and SP soils are poorly graded and contain little or no fines. When the terms GU or SU are used, uniform, i.e. closely graded, gravels or sands are indicated. Then the GP and SP symbols refer to non-uniform mix-

tures of coarse material and fine sand, with the intermediate particle sizes absent. Otherwise the GP and SP symbols cover all poorly graded coarse-grained soils lacking fines.

4·20 The GF and SF groups include all coarse-grained soils which contain excess fines (whether silty or clayey) as well as those poorly graded soils which contain a certain amount of binder.

4·21 There are no rigid boundaries between soil groups in the Casagrande system and boundary cases can be conveniently expressed by designations such as GW-SW or SC-SF. The former would imply a fairly equal distribution of gravel and sand sizes in the soil. The division between sandy gravels and gravelly sands is often taken to be where the soil contains 20 to 30 per cent of gravel sizes. The SC-SF soil would imply a well graded sand with only a slight excess of fines.

Identification of Fine-grained Soils

4·22 The plasticity chart for identifying fine-grained soils is shown in Fig. 4·1. Each soil is grouped according to the area of the chart in which the plotted point lies.

4·23 On the chart an empirical boundary known as the A-line separates inorganic clays from silty and organic soils. Although the latter two types overlap on the chart, they are normally easily differentiated by visual examination. At values of the liquid limit below 25 per cent, however, there is considerable overlapping. The inorganic silts (ML) cross the A-line and the lean clays extend into the area of the coarse-grained soils containing binder.

4·24 With experience these soils can be satisfactorily identified solely by visual and manual examination; otherwise soils may be quite adequately indicated by the use of boundary-line designations. The designations CL-SC, CL-SF, SF-ML or SF-SC may be adopted. The transition between coarse-grained and fine-grained soils is taken to be where there is rather more than 50 per cent of material finer than 0·1 mm.

4·25 Soils from a particular deposit may vary considerably in plasticity characteristics but it is usually found that the results for a number of individual samples lie on a straight line approximately parallel to the A-line.

4·26 Most inorganic clays lie fairly close to the A-line but some very tough clays may lie well above it, by an amount equivalent to a difference in plasticity index of over 10 per cent.

4·27 Kaolin clays usually lie below the A-line, behaving as inorganic silts and being classified as such (ML, MI, MH).

4·28 Some volcanic clays and bentonite clays lie just above the A-line at liquid limits of several hundred per cent.

4·29 The commonest inorganic silts have liquid limits less than 30 per cent and rock flour is usually non-plastic. Loess is also usually grouped as an ML soil. The presence of mica serves to increase the liquid limit of a soil whilst diatomaceous soils are generally non-plastic with liquid limits over 100 per cent.

4·30 Most organic soils, whether silts or clays, lie well below the A-line by an amount equivalent to a plasticity index of from 5 to 15 per cent and have a range in liquid limit of from 40 to over 100 per cent. Many of these soils are estuarine. Peats have very high liquid limits of several hundred per cent but a small plasticity index.

Field Identification Tests

4·31 Coarse-grained soils can be recognized fairly readily by field observation. Particle size can be judged visually with experience but a rapid sedimentation test may be found helpful. In this, some of the soil (excluding stones) is shaken up in a test tube full of water and allowed to settle. The coarser particles soon settle at the bottom and the proportions of finer materials can soon be gauged from the thickness of the succeeding layers and the turbidity of the water.

4·32 Fine-grained soils are distinguished by an examination to determine the following properties:—

(1) Dilatancy.

(2) Plasticity characteristics.

(3) Dry strength.

(4) Colour.

(5) Smell.

(6) Feel.

4·33 Dilatancy. A wet pat of soil is shaken in the palm of the hand and if the soil exhibits dilatancy it will show free water on the surface. This will disappear when the soil is squeezed between the fingers and the soil will become stiff and crumbly.

4·34 Rapid reaction to the test is characteristic of non-plastic uniform fine sands (SU), inorganic silts (ML) and diatomaceous soils (MH). With decreasing uniformity the reaction becomes more sluggish. Very little or no reaction to the shaking test is found with most inorganic clays (CL, CI, CH) and with highly plastic organic clays (OH).

4·35 Plasticity characteristics. A soft lump of the cohesive soil to be examined is kneaded into threads in the hand and the increase in stiffness noted as the plastic limit is approached; then the soil is re-formed into a lump and again kneaded until it crumbles, note being made of the toughness of the lump.

4·36 The higher the position of the soil on the plasticity chart with respect to the A-line, the stiffer are the threads near the plastic limit, and the tougher are the lumps below the plastic limit. Clays well above the A-line are stiff and tough. For glacial clays, which lie just above the A-line, the threads are fairly stiff but the lumps soon crumble below the plastic limit. Organic soils have weak, sometimes spongy, threads and lumps which crumble easily.

4·37 Dry strength. A piece of air-dried soil is crushed between the fingers and its resistance to crushing observed.

4·38 Table 4·2 gives the typical dry strength characteristics of various soils.

TABLE 4·2

DRY STRENGTH OF VARIOUS SOILS

Dry strength	Soil types
None	Non-plastic ML, MI and MH soils.
Low	Soils of low plasticity below the A-line. Some very silty clays just above the A-line (CL).
Medium	Most CL and CI soils. CH, MH and OH soils near the A-line.
High	Most CH soils, CL and CI soils well above the A-line. Some OH soils near the A-line.
Very high	CH soils well above the A-line.

4·39 COLOUR. In general, dark colours of grey, brown or black indicate organic soils, whereas bright colours are usually found with inorganic soils.

4·40 SMELL. Organic soils of the OL and OH groups commonly have a distinctive smell. This method of identification should be applied if possible to fresh samples.

4·41 FEEL. The sense of touch can be used to distinguish silts from clays and from sands. Whereas sand has a gritty feel, silt has a rough (though not gritty) texture and clay has a smooth greasy feel. Also, clay sticks to the fingers and dries slowly but silt dries fairly quickly and can be dusted off the fingers leaving only a stain.

U.S. PUBLIC ROADS ADMINISTRATION CLASSIFICATION (1945)

4·42 The revised U.S. Public Roads Administration classification was adopted by the U.S. Highway Research Board in 1945[6]. The original system was developed by the U.S. Bureau of Public Roads[7] [8] and published in its final form by the then renamed U.S. Public Roads Administration in 1942[9].

4·43 The original method of classification is shown in Table 4·3 and Fig. 4·2. It contained eight groups ranging from A-1, a well graded gravel-sand-clay, to A-8, a peat. The system was based on the following six soil properties:—

Particle-size distribution.

Liquid limit.

Plasticity index.

Shrinkage limit.

Field moisture equivalent.

Centrifuge moisture equivalent.

The tests were made on soil passing the No. 10 U.S. sieve.

TABLE 4·3

ORIGINAL U.S. PUBLIC ROADS ADMINISTRATION (P.R.) SYSTEM OF CLASSIFICATION (1942 VERSION)

		Group	A—1	A—2		A—3	A—4	A—5	A—6	A—7	A—8
				A—2.F	A—2.P						
Particle-size distribution		Coarse material (%) Retained on No. 10 sieve (%)	0—65	—	—	—	—	—	—	—	—
	Soil mortar	Total sand (%)	70—85	55 min.	55 min.	—	55 maximum				
		Coarse sand (%)	45—60	—	—	—	—	—	—	—	—
		Silt (%)	10—20	—	—	—	—	—	—	—	—
		Clay (%)	5—10	—	—	—	—	—	—	—	—
		Passing No. 200 sieve (%)	—	—	—	0—10	—	—	—	—	—
Soil constants		Liquid limit (%)	14—35	35 max.	35 max.	NP	20—40	35 minimum			
		Plasticity index (%)	4—9	NP—3	3—15	NP	0—15	See Fig. 4·2			
		Shrinkage limit (%)	14—20	—	—	—	20—30	See Fig. 4·2			
		Field moisture equivalent (%)	—	—	—	—	30 max.	See Fig. 4·2			
		Centrifuge moisture equivalent (%)	15 max.	25 max.	25 max.	12 max.	—	—			

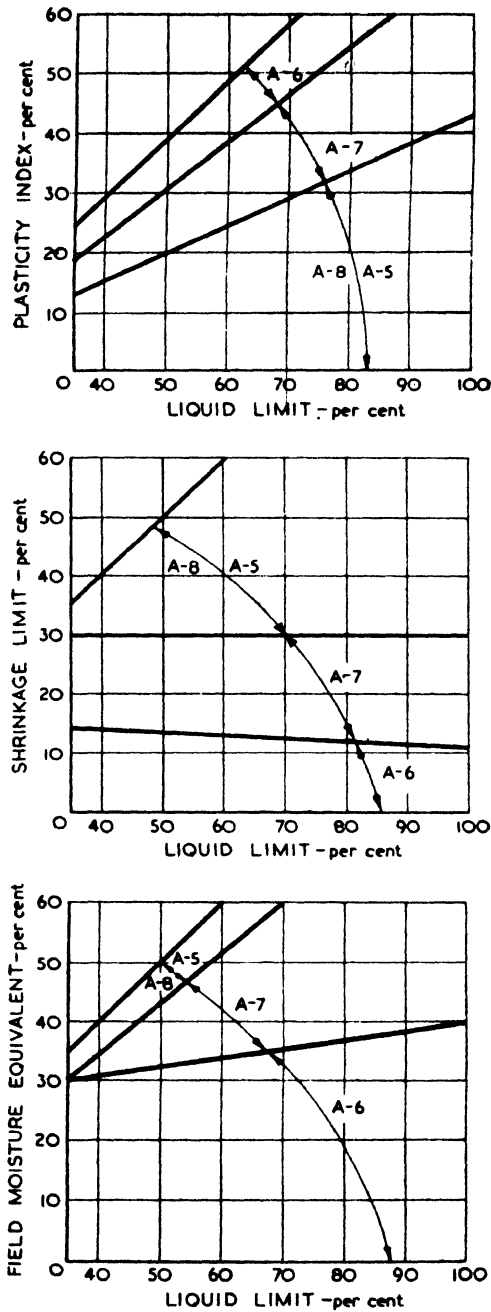



FIG. 4·2 CHART OF SOIL CHARACTERISTICS FOR IDENTIFICATION OF FINE-GRAINED SOILS BY ORIGINAL U.S. PUBLIC ROADS ADMINISTRATION (P.R.) SYSTEM (1942 VERSION)

4·44 In the revised classification the groupings were slightly changed, the necessary tests reduced to the first three mentioned above, sub-groupings formed and a new system of group-indexing introduced. The purpose of these changes was to make the system suitable for subgrade classification, to make it correspond more closely with soil types in the field and to aid visual identification. A major difference is that in the revised system the whole soil is considered and not merely the soil mortar as in the original system.

4·45 The most noticeable change is the transfer of clean gravels and coarse sands from the A-3 group into the A-1 group, in a recognition of their superior value to fine sands as a subgrade material.

4·46 The revised system comprises seven groups, A-1 to A-7, but can be further subdivided into 12 sub-groups. Table 4·4 shows the group classification and Table 4·5 shows the classification into sub-groups.

4·47 These tables are self-explanatory except for the meaning of the group index. This index is a means of rating the value of a soil as a subgrade material within its own group. It is not used in order to place a soil in a particular group; that is done directly from the results of sieve analysis, and the liquid limit and plasticity index. The higher the value of the index, the poorer is the quality of the material. The group index is a function of the amount of material passing the No. 200 sieve, the liquid limit and the plasticity index, and can be determined from the charts or formula given in Fig. 4·3.

4·48 The grouping of the A-4 to A-7 soils is conveniently illustrated in Fig. 4·3. The A-line of the Casagrande plasticity chart has been superimposed on the diagram. The actual materials included in the various groups and sub-groups are given below.

Granular materials

4·49 Group A-1. The typical material of this group is a well graded mixture of stone fragments or gravel, coarse sand, fine sand and a non-plastic or feebly plastic soil binder. However, this group includes also stone fragments, gravel, coarse sand, volcanic cinders, etc.

4·50 Sub-group A-1-a includes materials consisting predominantly of stone fragments or gravel, either with or without a well graded binder of fine material.

4·51 Sub-group A-1-b includes those materials consisting predominantly of coarse sand, either with or without a well graded soil binder.

4·52 Group A-3. The typical material of this group is fine beach sand or fine desert blown sand, without silty or clay fines or with a very small amount of non-plastic silt. The group includes also stream-deposited mixtures of poorly graded fine sand and limited amounts of coarse sand and gravel.

4·53 Group A-2. This group includes a wide variety of granular materials which are border-line cases between the materials falling in groups A-1 and A-3 and the silt-clay materials of groups A-4, A-5, A-6 and A-7. It includes all materials containing 35 per cent or less passing a No. 200 sieve* which cannot be classified as A-1 or A-3, owing to fines content or plasticity, or both, in excess of the limitations for those groups.

*All sieves mentioned in connexion with the P.R. classification are in American sizes.

TABLE 4·4

REVISED U.S. PUBLIC ROADS ADMINISTRATION (P.R.) SYSTEM (1945): CLASSIFICATION INTO GROUPS

General classification	Granular materials (35% or less passing No. 200)			Silt-clay materials (More than 35% passing No. 200)			
Group classification	A—1	A—3*	A—2	A—4	A—5	A—6	A—7
Sieve analysis, percentage passing:							
No. 10							
No. 40	50 max.	51 min.					
No. 200	25 max.	10 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40 sieve							
Liquid limit (%)				40 max.	41 min.	40 max.	41 min.
Plasticity index (%)	6 max.	N.P.		10 max.	10 max.	11 min.	11 min.
Group index			4 max.	8 max.	12 max.	16 max.	20 max.
General rating as a subgrade	Excellent to good			Fair to poor			

CLASSIFICATION PROCEDURE: With required test data available, proceed from left to right on above chart and correct group will be found by process of elimination. The first group from the left into which the test data will fit is the correct classification. (Note: All limiting test values are shown as whole numbers. If fractional numbers appear on test reports, convert to nearest whole number for purposes of classification.)

*The placing of A—3 before A—2 is necessary in the left-to-right elimination process and does not indicate superiority of A—3 over A—2.

TABLE 4·5

REVISED U.S. PUBLIC ROADS ADMINISTRATION (P.R.) SYSTEM (1945): CLASSIFICATION INTO SUB-GROUPS

General classification	Granular materials (35% or less passing No. 200 sieve*)							Silt-clay materials (more than 35% passing No. 200 sieve*)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	•A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percentage passing: No. 10 sieve* No. 40 sieve* No. 200 sieve*	50 max. 30 max. 15 max.	50 max. 25 max.	51 min. 10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40 sieve* Liquid limit (%) Plasticity index (%)	6 max.		N.P.	40 max. 10 max.	41 min. 10 max.	40 max. 11 min.	41 min. 11 min.	40 max. 10 max.	41 min. 10 max.	40 max. 11 min.	41 min. 11 min.[a]
Group index[b]	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as a sub-grade	Excellent to good					Fair to poor					

CLASSIFICATION PROCEDURE: With required test data available, proceed from left to right on above chart and correct group will be found by process of elimination. The first group from the left into which the test data will fit is the correct classification.

[a]Plasticity index of A-7-5 sub-group is equal to or less than liquid limit minus 30. Plasticity index of A-7-6 sub-group is greater than liquid limit minus 30.

[b]See group index formula Fig. 4·3 for method of calculation. Group index should be shown in parentheses after group symbol as:—A-2-6 (3), A-4 (5), A-6 (12), A-7-5 (17), etc.

*Sieve sizes are American.
Approximate British equivalents as follows.

United States A.S.T.M. E.11/26	Great Britain B.S. 410
10	8
40	36
200	200

PER CENT PASSING No. 200 SIEVE

GROUP INDEX = 0·2a + 0·005ac + 0·01 bd

where:-

a = That portion of percentage passing No. 200 sieve greater than 35 and not exceeding 75 expressed as a positive whole number (0 to 40)

b = That portion of percentage passing No. 200 sieve greater than 15 per cent and not exceeding 55 per cent expressed as a positive whole number (0 to 40)

c = That portion of the numerical liquid limit greater than 40 and not exceeding 60 expressed as a positive whole number (0 to 20)

d = That portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20)

Group index = Sum of readings on vertical scale of charts

Chart for determination of group index

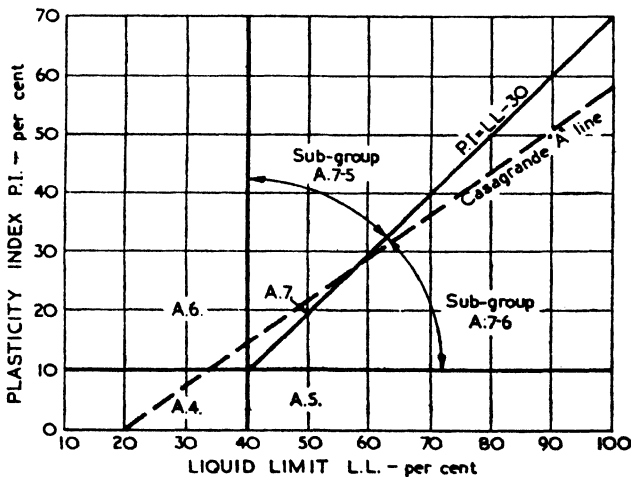



FIG. 4·3 CHARTS FOR USE WITH REVISED PUBLIC ROADS (P.R.) CLASSIFICATION

Liquid limit and plasticity index ranges for silt-clay soils

4·54 Sub-groups A-2-4 and A-2-5 include various granular materials containing 35 per cent or less passing a No. 200 sieve and with the portion passing a No. 40 sieve having the characteristics of the A-4 and A-5 groups. They include such materials as gravel and coarse sand with silt content or plasticity index in excess of the limitations of group A-1, and fine sand with non-plastic silt content in excess of the limitations of group A-3.

4·55 Sub-groups A-2-6 and A-2-7 include materials similar to those described under sub-groups A-2-4 and A-2-5, except that the fine portion contains plastic clay having the characteristics of the A-6 or A-7 group. The approximate combined effects of plasticity indices in excess of 10 per cent and percentages passing a No. 200 sieve in excess of 15 are reflected by the group index values of from 0 to 4.

Silt-clay materials

4·56 GROUP A-4. The typical material of this group is a non-plastic or moderately plastic silty soil usually having 75 per cent or more passing a No. 200 sieve. The group includes also mixtures of fine silty soil and up to 64 per cent of sand and gravel retained on a No. 200 sieve. The group index values range from 1 to 8, with increasing percentages of coarse material being reflected by decreasing group index values.

4·57 GROUP A-5. The typical material of this group is similar to that described under group A-4, except that it is usually of diatomaceous or micaceous character and may be highly elastic as indicated by the high liquid limit. The group index values range from 1 to 12, with increasing values indicating the combined effect of increasing liquid limits and decreasing percentages of coarse material.

4·58 GROUP A-6. The typical material of this group is a plastic clay soil usually having 75 per cent or more passing a No. 200 sieve. The group includes also mixtures of fine clayey soil and up to 64 per cent of sand and gravel retained on a No. 200 sieve. Materials of this group usually have a high volume change between wet and dry states. The group index values range from 1 to 16, with increasing values indicating the combined effect of increasing plasticity indices and decreasing percentages of coarse material.

4·59 GROUP A-7. The typical material of this group is similar to that described under group A-6, except that it has the high liquid limits characteristic of the A-5 group and may be elastic as well as subject to high volume change. The range of group index values is from 1 to 20, with increasing values reflecting the combined effect of increasing liquid limits and plasticity indices and decreasing percentages of coarse material.

4·60 Sub-group A-7-5 includes those materials with moderate plasticity indices in relation to liquid limit, which may be highly elastic as well as subject to considerable volume change.

4·61 Sub-group A-7-6 includes those materials which have high plasticity indices in relation to their liquid limits and which are subject to extremely high volume changes.

TABLE 4·6

ORIGINAL CIVIL AERONAUTICS ADMINISTRATION (C.A.A.) CLASSIFICATION SYSTEM (1944)

Soil	Material passing No. 10 U.S. sieve			Material passing No. 40 U.S. sieve			Capillary rise of minus No. 10 material (in.)	California bearing ratio (soaked) (%)	Subgrade and sub-base classification			
	Sand (%)	Silt (%)	Clay (%)	Liquid limit (%)	Plasticity index (%)	Volume change at FME (%)			No frost; good drainage	Severe frost; good drainage	No frost; poor drainage	Severe frost; poor drainage
E-1	>85	0–10	0–5	<25	0–6	0–6	0–12	>20	F_a R_{1a}	F_a R_{2a}	F_a R_{2a}	F_a R_{2a}
E-2	>75	0–15	0–10	<25	0–6	0–6	0–36	>20	F_a R_{1a}	F_a R_{2a}	F_a R_{1a}	F_a R_{2a}
E-3	>55	10–40	0–20	<35	0–10	0–10	>36	>18	F_a R_{1a}	F_1 R_{2a}	F_2 R_{1a}	F_2 R_{2a}
E-4	>55	10–30	5–25	<45	5–15	5–15	>36	13–40	F_1 R_{1a}	F_2 R_{2b}	F_3 R_{1a}	F_4 R_{2b}
E-5	<65	20–75	0–20	<45	0–10	0–15	>36	9–20	F_2 R_{1a}	F_2 R_{2b}	F_4 R_{2a}	F_6 R_{2b}
E-6	<55	5–70	10–40	<50	10–30	10–30	>36	6–12	F_3 R_{1b}	F_3 R_{2b}	F_6 R_{2b}	F_7 R_{2c}
E-7	<55	5–70	15–50	<60	15–40	20–40	>36	4–8	F_4 R_{1b}	F_4 R_{2b}	F_7 R_{2c}	F_8 R_{2c}
E-8	<55	5–50	>30	>70	20–50	30–50	<36	3–5	F_5 R_{2b}	F_7 R_{2c}	F_8 R_{2c}	F_9 R_{2d}
E-9	<55	5–50	>30	>80	30–60	40–60	<36	2–4	F_6 R_{2b}	F_8 R_{2c}	F_9 R_{2d}	F_{10} R_{2d}
E-10	<55	30–80	<30	<60	0–25	—	>36	1–3	F_8 R_{2c}	F_9 R_{2d}	F_{10} R_{2d}	F_{10} R_{2d}

Notes: (1) FME denotes Field Moisture Equivalent.
(2) The F and R symbols indicate the relative suitability of the various soil groups for use in the subgrade or sub-base under flexible and rigid pavements respectively, for the conditions stated.

CIVIL AERONAUTICS ADMINISTRATION (C.A.A.) CLASSIFICATION SYSTEM

4·62 This system[10] is based upon the particle-size analysis, plasticity characteristics, expansive qualities and California bearing ratio of soils. Its use is limited to a subgrade classification for airfield pavement design. For this purpose the C.B.R. test was introduced into the classification system but it sometimes contradicts the grouping of a particular soil as determined by its other physical characteristics. The soil is then placed in the lower group or an interpolation is made.

4·63 The classification is set out in Table 4·6. The groups decrease in subgrade value from E-1 to E-10. It will be noticed that the classification is based on tests on the soil mortar. Field conditions are allowed for in the last four columns, where the value of each soil group is assessed in terms of its behaviour under the action of frost or excessive moisture and given an appropriate symbol.

4·64 The following list describes the C.A.A. soil groups with reference to equivalent groups in the P.R. classification.

- E-1 a free-draining non-plastic sand such as A-3.
- E-2 a sand with slightly more binder than E 1.
- E-3 a non-plastic or moderately plastic A-2 soil.
- E-4 a plastic A-2 soil.
- E-5 a non-plastic or moderately plastic A-4 silt.
- E-6 a more plastic A-4 silt, or A-4 or A-6 silty clays, or A-6 or A-7 clays of low plasticity.
- E-7 A-6 or A-7 clays of average plasticity.
- E-8 A-6 or A-7 clays of high plasticity.
- E-9 A-6 or A-7 clays of very high plasticity.
- E-10 highly elastic soil such as A-5.

4·65 The above classification system was established in 1944, but in 1946 a new edition was published introducing a number of subdivisions into the system and altering the test limits of some of the groups. It forms a " Revised C.A.A. System " and is shown in Table 4·7.

4·66 All sieve sizes quoted in the C.A.A. classification charts refer to American standard sizes.

4·67 In Fig. 4·4 the Civil Aeronautics system is approximately correlated with the Casagrande classification and the Public Roads Administration system on the basis of bearing capacity as suggested by Fruhauf[11].

COMPACTION CLASSIFICATION

4·68 This system is an example of the way in which soil classification can be very much simplified where only a limited area of soil of similar characteristics is being examined. It was devised by K. B. Woods[12], for use in the Mississippi basin where the soils are very uniform over large areas and where differences in type could conveniently be correlated with the maximum dry density as determined by the standard compaction test.

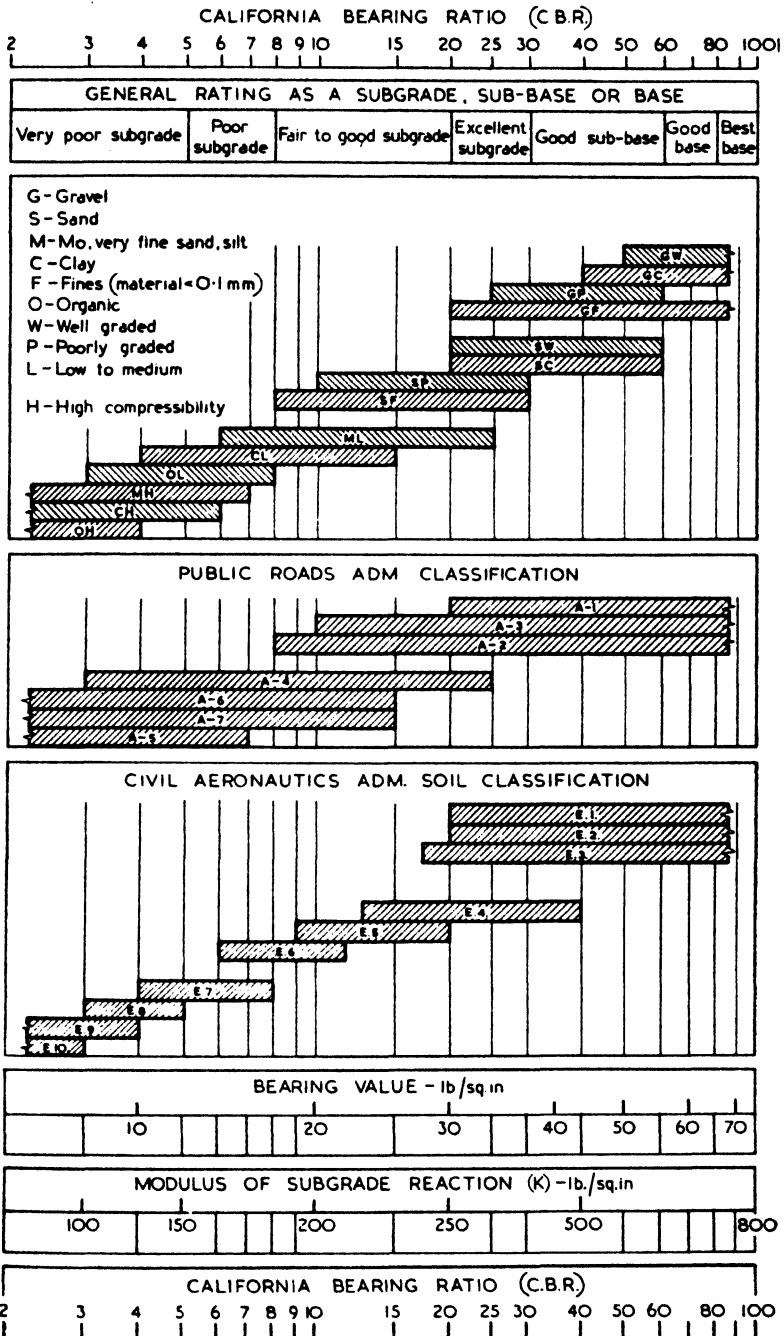



FIG. 4·4 APPROXIMATE CORRELATION OF THE CASAGRANDE, P.R. AND C.A.A. CLASSIFICATIONS ON THE BASIS OF BEARING CAPACITY

4·69 Woods' system simply divides soils into groups as shown in Table 4·8.

TABLE 4·8

COMPACTION CLASSIFICATION

Maximum dry density (lb./cu. ft)	General value as a foundation
over 130	excellent
120 - 130	good
110 - 120	fair
100 - 110	poor
70 - 100	very poor

The system cannot be used everywhere since it depends on a determination of a single soil characteristic which, though often significant, cannot be expected to take account of all the factors of importance in judging the behaviour of a soil.

BURMISTER'S METHOD OF DESCRIPTIVE SOIL CLASSIFICATION

4·70 Burmister[13] has developed a system of classification based on a number of definitions of soil components (Table 4·9) devised by the American Society for Engineering Education (A.S.E.E.)[14]. The chief point of interest in these definitions is the use of the term " clay-soil " instead of " clay." This recognizes the fact that the silt fraction of a soil cannot be separated easily from the clay fraction in the field in order to determine the proportions of each. The coarser-grained soil fractions can be distinguished visually.

4·71 Burmister has suggested a method of arriving at a series of composite soil names which are based on these definitions (Table 4·10). Examples are given of composite soil names determined by this method.

4·72 In determining the appropriate composite soil names in the field Burmister suggests the use of large standard bottled samples of variously proportioned mixtures of the cohesionless soils. These provide a means of comparison, when estimating the proportions of each soil component. He suggests making up at least seven dry gravel-sand mixtures, three dry sand samples and four moist sand-silt mixtures.

4·73 All identification is carried out visually or with the usual simple manual tests, the proportion of coarser components being estimated before the proportion of finer ones. In carrying out manual tests on cohesive soils, the gravel fraction is first removed by hand and the plasticity or other property examined on the overall sand/clay-soil fraction.

TABLE 4·9

AMERICAN SOCIETY FOR ENGINEERING EDUCATION: RECOMMENDATIONS FOR SOIL COMPONENTS AND FRACTIONS

Principal components	Description	Sieve limit	Sieve sizes for sub-components		
			Coarse	Medium	Fine
Boulders and rock*	Retained on 3-in. sieve	Lower			3 in.
Gravel and stone	Passes 3-in. sieve; retained on No. 10 sieve	Upper Lower	3 in. 1 in.	1 in. $\frac{3}{8}$ in.	$\frac{3}{8}$ in. No. 10
Sand	Passes No. 10 sieve; retained on No. 200 sieve	Upper Lower	No. 10 No. 30	No. 30 No. 60	No. 60 No. 200
Silt	Passes No. 200 sieve; non-plastic, little or no strength when air-dried	Upper	No. 200		
Clay-soil	Passes No. 200 sieve; exhibits plastic properties and clay qualities within a certain range of moisture content; considerable strength when air-dried.	Upper	No. 200		

*Boulders and gravel refer to waterworn material; rock and stone refer to angular fragments.

TABLE 4·10

THE BURMISTER DESCRIPTIVE CLASSIFICATION USING COMPOSITE SOIL NAMES

Descriptive terms for cohesionless soil to be used in forming the soil name

Soil component	As written in the soil name	Descriptive or qualifying terms as written	Range of proportions
Principal	Gravel, sand, silt	and	50% or more* 35% to 50%
Others	Gravel, sand, silt	some, little, trace	20% to 35%* 10% to 20%* 1% to 10%*
Subcomponents		coarse to fine coarse to medium medium to fine coarse medium fine	all sizes 10% fine 10% coarse 10% medium fine 10% coarse and fine 10% coarse and medium

Additional descriptive terms:

1. Colour, grain shape, etc.
2. Degree of compactness, degree of plasticity.
3. Inorganic constituents (mica, shells, and foreign matter).
4. Organic matter (roots, humps, peat, and muck).
5. Geological origin (alluvial, glacial, wind, beach, swamp, etc.), also horizon.

*Finer than, or coarser than, the principal soil component.

Descriptive names of clay soils based on degree of plasticity*

Degree of plasticity	Plastic index (%)	Descriptive name as written	Qualities
Non-plastic	0–1	silt	Friable
Slight plasticity	1–5	trace CLAY	Desirable
Low plasticity	5–10	little CLAY	Cohesiveness
Medium plasticity	10–20	CLAY and SILT	Increasingly objectionable
High plasticity	20–35	silty clay	plastic displacement
Very high plasticity	>35	CLAY	and compressibility

*Overall plasticity of sand/silt/clay-soil fraction.

EXAMPLES OF COMPOSITE SOIL NAMES.

1. "Light brown coarse to fine GRAVEL, and coarse to fine SAND, trace silt."
2. "Light tan coarse to fine GRAVEL, some coarse to fine sand, trace silt."
3. "Light brown medium to fine SAND, little silt"; medium compact.
4. "Brown medium to fine SAND, some silt, trace clay"; slight plasticity.
5. "Grey-brown SILT, some medium to fine sand, little clay"; low plasticity.
6. "Grey CLAY and SILT, little medium to fine sand"; medium plasticity.

TEXTURAL CLASSIFICATIONS

4·74 Textural classifications are based exclusively on the particle-size distribution of soils. The usual method is to classify soils on a triangular diagram which relates the percentages of the sand, silt and clay fractions in the soil. Fig. 4·5 shows one such diagram adopted by the U.S. Bureau of Soils[15] [16]. The diagram is divided into ten areas, to each of which is given a general name which roughly describes the soil type.

4·75 Textural classifications are mainly of value for describing coarse-grained soils; they are not so suitable for classifying clay soils whose properties are less dependent on the particle-size distribution.

4·76 In using a textural classification, care should be taken to note the particle-size scale on which it is based. Fig. 4·6 shows the principal particle-size scales, and Fig. 4·7 the corresponding British and American sieve sizes. The best-known scales used in engineering are the Continental scale, the U.S. Public Roads Administration scale and the Massachusetts Institute of Technology scale which has since been adopted as the British Standard[17] and which is the basis of the Casagrande classification.

GEOLOGICAL MAPS AND MEMOIRS

4·77 The 1-in. and 6-in. maps of Great Britain[18] and the Geological Survey memoirs[19] are of considerable value to the engineer.

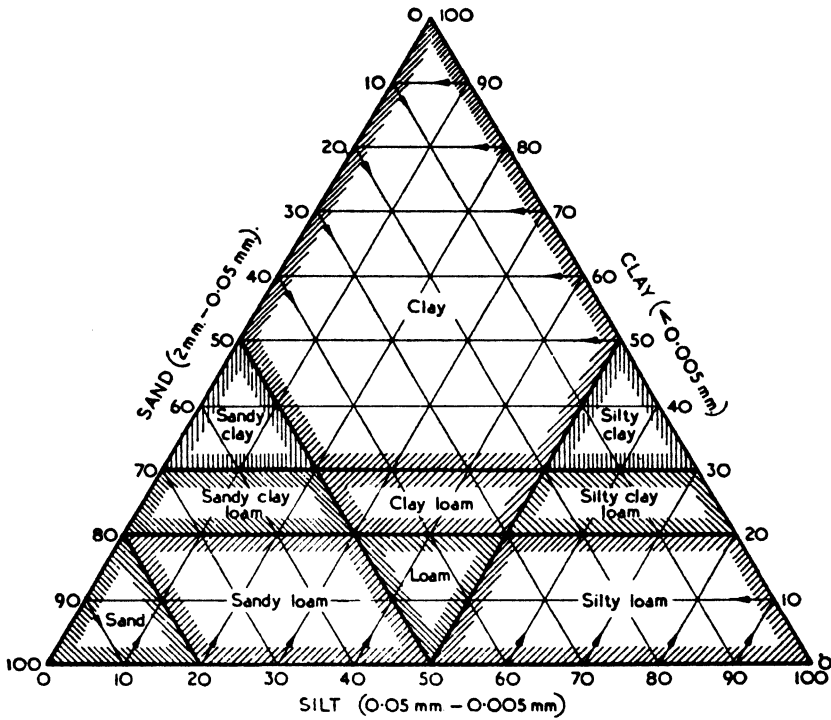
4·78 Either solid geology or drift maps are obtainable, depending on the area concerned. The solid geology maps do not show glacial deposits; all surface deposits are shown in the drift maps, which are therefore more suitable for investigations of shallow sites. The memoirs are also of value since they give detailed explanations of the 1-in. scale maps.

CLASS	% SAND	% SILT	% CLAY
Sand	80–100	0– 20	0– 20
Sandy loam	50– 80	0– 50	0– 20
Loam	30– 50	30– 50	0– 20
Silty loam	0– 50	50–100	0– 20
Sandy clay loam	50– 80	0– 30	20– 30
Clay loam	20– 50	20– 50	20– 30
Silty clay loam	0– 30	50– 80	20– 30
Sandy clay	50– 70	0– 20	30– 50
Clay	0– 50	0– 50	30–100
Silty clay	0– 20	50– 70	30– 50

FIG. 4·5 A TYPICAL TEXTURAL CLASSIFICATION CHART (U.S. BUREAU OF SOILS SYSTEM)

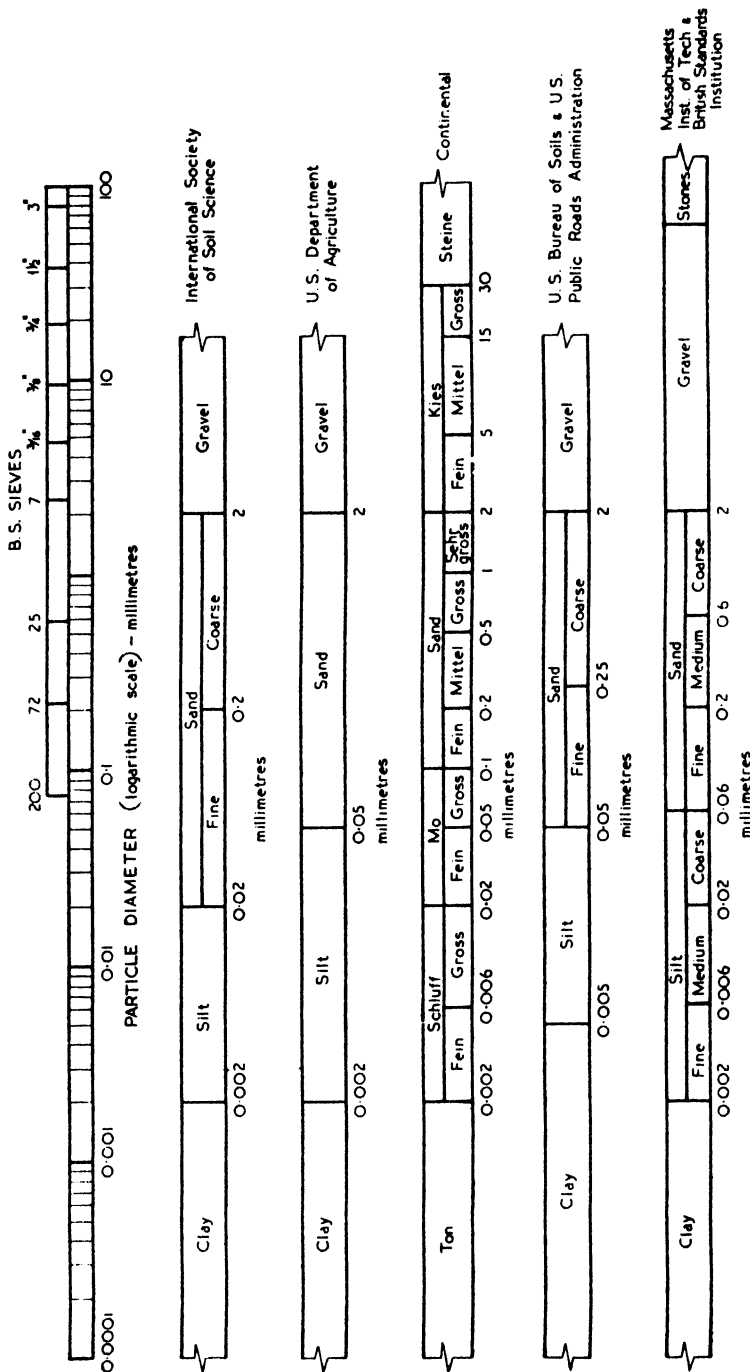



FIG. 4·6 PRINCIPAL PARTICLE-SIZE SCALES

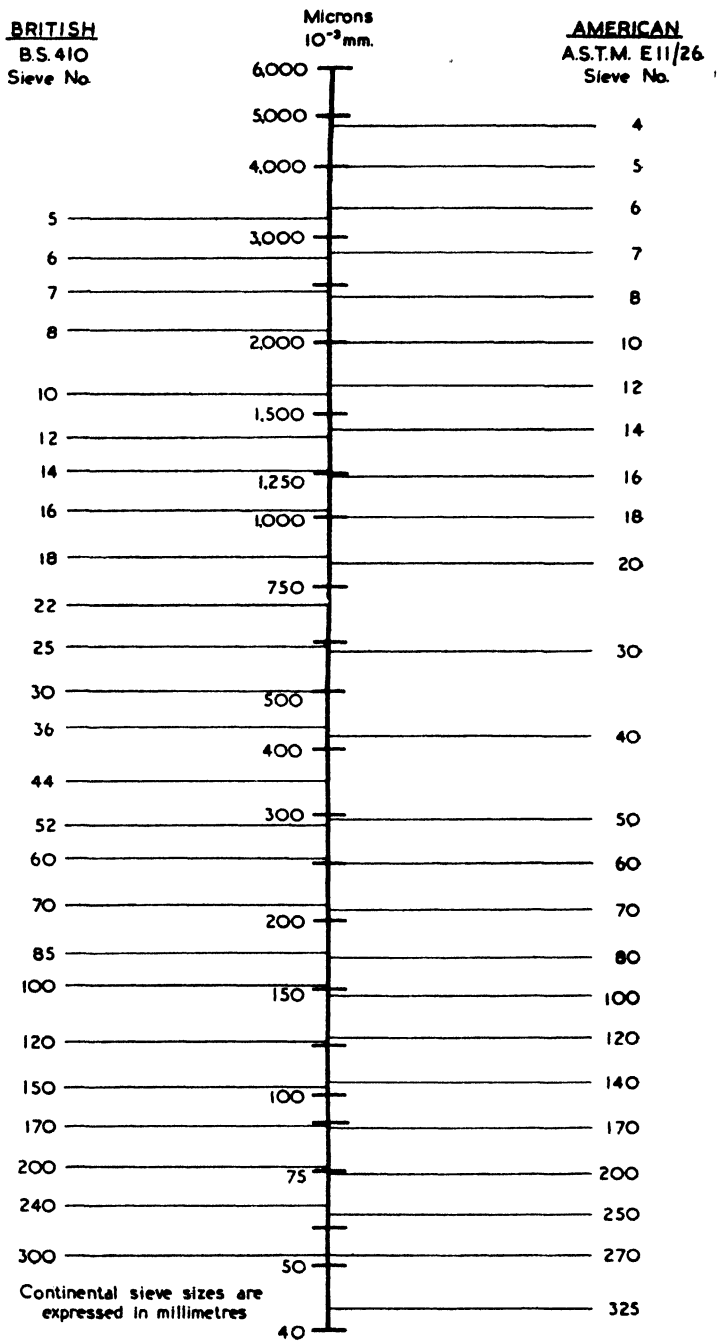



FIG. 4·7 COMPARISON OF BRITISH, AMERICAN AND CONTINENTAL SIEVE SIZES

SUMMARY

4·79 This chapter describes the following methods of classifying soils:—

(1) Casagrande classification and an extended form of this system which has been adopted by the Road Research Laboratory.
(2) U.S. Public Roads Administration classification (original and revised).
(3) Civil Aeronautics Administration classification (original and revised).
(4) Compaction classification.
(5) Burmister classification.
(6) Textural classification.

4·80 Reference is made to the geological maps and memoirs covering Great Britain.

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   General Memoirs   Economic Memoirs
   District Memoirs   Sheet Memoirs.

CHAPTER 5

CHEMICAL TESTS FOR SOIL

INTRODUCTION

5·1 Although there is at present only a limited amount of information on the influence of chemical composition on those properties of soil of concern to the civil engineer, it is known that the presence of some constituents, such as organic matter, chalk and sulphates may influence certain of these properties, particularly those of interest in soil stabilization work. Certain quantitative tests are therefore employed to estimate the proportions of these constituents in soils, as well as to determine the acidity or alkalinity (pH value) of the soil water. These tests are not routine classification tests like particle-size analyses and plasticity tests, but have their main applications in specific problems in which the chemical composition of the soil is believed to be a factor. These quantitative tests are largely methods of analysis developed for agriculture[1] [2]. A brief description is given of the laboratory techniques used at the Road Research Laboratory.

ORGANIC MATTER

5·2 Organic matter is an undesirable constituent of the soil from the engineering point of view, since it reduces the bearing capacity, and swells or shrinks when the moisture content or the applied load changes. The origin and composition of organic matter in soils are discussed more fully in Chapter 2.

Methods of determining the Organic Matter Content

5·3 Although it is possible to obtain a relative colorimetric estimate of the amount of humus in a soil by extraction with alkali, this method will not indicate the total amount of organic matter present, i.e. including undecomposed plant remains. Waksman and Stevens[3] consider that the most reliable method is to determine the percentage of organic carbon, and to multiply this by a factor of 1·724 to give the total organic matter content. This assumes that soil organic matter contains on an average 58 per cent of carbon.

5·4 Approximate determinations of the organic matter content can be made by finding the loss in weight of the soil after treatment with hydrogen peroxide, or after ignition at high temperatures (700-800°C.), but both these methods have disadvantages. While treatment with peroxide appears to remove the humus and colloidal organic matter, it has only a limited action on undecomposed plant remains such as roots and fibres, and an accurate quantitative determination is difficult. In the ignition method all the organic material is destroyed, but the high temperatures involved also break down the hydrated alumino-silicates in the clay fraction, and the carbonates. The values obtained by this method with clay or chalky soils are therefore in error on the high side.

5·5 A more accurate estimation of the organic matter content of a soil can be made by oxidizing the organic carbon quantitatively, which can be done

by either wet or dry oxidation methods. At the Road Research Laboratory use is made of a wet method, due to Schollenberger[4] [5] and modified by Walkley and Black[6], in which the soil is treated with chromic acid in hot sulphuric acid, the excess chromic acid remaining after oxidation of the carbon being estimated quantitatively by ferrous sulphate. This method is not affected by the presence of carbonates in the soil, and it is also able to discriminate between elementary carbon, such as coal, charcoal and graphite, and the soil organic matter proper. Chlorides interfere with the estimation, since they are oxidized by chromic acid with the liberation of free chlorine, but a correction may be made if their concentration has previously been determined, since they react quantitatively.

5·6 The most accurate method of estimating organic carbon appears to be by dry combustion, in which a sample of the soil is heated to about 900°C. and the organic carbon is oxidized by a stream of dried and purified air or oxygen. Carbon dioxide is formed and absorbed in a suitable absorbent, e.g. soda-lime, and weighed. The weight of carbon removed from the soil is then calculated from the weight of carbon dioxide so obtained. Carbonates cause trouble in this method, since they evolve carbon dioxide when heated to high temperatures, and the inorganic carbon in chalky soils must therefore be removed before the estimation, e.g. by treatment with sulphurous acid, or determined separately and a suitable correction made. In addition, the dry combustion method does not distinguish between the soil organic matter and elementary carbon.

5·7 The disadvantages of the dry combustion method are that the equipment is more complicated and expensive than that required for the wet oxidation method, and that only one sample can be dealt with in the space of about an hour. It is therefore considered to be unsuitable for routine testing in a soil mechanics laboratory, and is not described in detail.

5·8 In the following paragraphs, descriptions are given of the experimental techniques involved in the first three of the methods referred to above, viz. the loss-on-ignition method, the peroxide oxidation method and the dichromate oxidation method. All three methods can be usefully applied in soil mechanics work, provided that their limitations are borne in mind. Thus, the loss-on-ignition method can be used on sandy soils containing little or no clay or chalky material. The peroxide oxidation method may be used in conjunction with the British Standard procedure for particle-size analysis, and after treatment the soil sample may be subjected to the sieving and sedimentation techniques used in this analysis. The dichromate oxidation method is considered to be the most suitable for the majority of soils. It is also fairly rapid and a number of tests may be run concurrently, while the apparatus required (Plate 5·1A) is relatively simple and inexpensive.

Determination of the Organic Matter Content of Soil: Loss-on-ignition Method

5·9 The apparatus required is as follows:—

(1) Thermostatically controlled drying oven, set to 105-110°C.

(2) Chainomatic balance, accurate to 0·01 gm.

(3) Silica or porcelain crucible of about 30-ml. capacity.

(4) Méker gas burner, pipe-clay triangle, tripod, etc.

5·10 The dry crucible is first weighed accurately to the nearest 0·01 gm and approximately 20 gm. of oven-dried soil are placed in it. The crucible and contents are then weighed accurately to the nearest 0·01 gm and the weight of soil calculated by difference. The soil is then ignited by heating the crucible to red heat over the Méker burner, care being taken to avoid loss of soil due to currents of the burning gas. About 20 to 30 minutes' heating should be sufficient to destroy the organic matter.

5·11 The crucible and the contents are cooled, and again weighed accurately to the nearest 0·01 gm and the loss in weight due to ignition is determined by difference. This loss is reported as the organic matter content to the nearest 0·1 per cent based on the total oven-dried weight of the soil.

Determination of the Organic Matter Content of Soil: Peroxide Oxidation method

5·12 The apparatus and reagents required are as follows:—

(1) Thermostatically controlled drying oven, set to 105-110°C.

(2) Chainomatic balance, accurate to 0·01 gm.

(3) Porcelain evaporating dish, 14 cm. in diameter.

(4) Thermometer, reading over the range 0 - 100°C., filter funnel, filter papers, and 400-ml. beaker.

(5) 6 per cent (20-volume) solution of hydrogen peroxide.

5·13 Between 50 and 100 gm of oven-dried soil are weighed out to the nearest 0·01 gm. and placed in the evaporating dish, and about 100 ml. of the hydrogen peroxide solution are added. The dish and contents are then warmed gently over a bunsen flame until the temperature is about 60°C. while the solution is stirred with a glass rod in order to release bubbles of gas from the soil and to permit the peroxide to react completely with the organic matter. The reaction should be allowed to continue until gas is no longer generated at a very rapid rate. With highly organic soils, additional quantities of peroxide solution may be required to complete the oxidation, which may take a day or more. When the reaction is complete the excess peroxide must be removed by boiling the solution for 10 to 15 min.

5·14 The soil is then filtered, washed with distilled water and dried in the oven. The dry soil is then weighed to the nearest 0·01 gm and the loss in weight determined. This loss is then reported to the nearest 0·1 per cent based on the original oven-dried weight of the soil.

Determination of the Organic Matter Content of Soil: Dichromate Oxidation method

5·15 The apparatus and reagents required are as follows:—

(1) Thermostatically controlled drying oven, set to 105 - 110°C.

(2) Analytical balance accurate to 0·001 gm.

(3) Burettes, pipettes, volumetric flasks and graduated measuring cylinders.

(4) N(Normal) solution of potassium dichromate, made by dissolving 49·05 gm of the salt (A.R. grade) in 1 litre of distilled water.

(5) N(Normal) solution of ferrous sulphate, made by dissolving 277·00 gm of the salt (A.R. grade) in 1 litre of 0·5 N sulphuric acid (the latter may be prepared by diluting 14 ml. of the concentrated acid to 1 litre with distilled water).

(6) Concentrated sulphuric acid.

(7) 85 per cent phosphoric acid.

(8) 0·5 per cent solution of diphenylamine in concentrated sulphuric acid.

5·16 It is first necessary to standardize the ferrous sulphate solution in terms of the dichromate solution. To do this, exactly 10 ml. of the dichromate solution are run into a 500-ml. conical flask from a burette, and 20 ml. concentrated sulphuric acid are added. The mixture is shaken and allowed to cool for some minutes, after which 200 ml. of distilled water are added, followed by 10 ml. of phosphoric acid and 1 ml. of diphenylamine solution and the mixture is again shaken. Ferrous sulphate solution is then run from a second burette in 0·5-ml. increments until the colour of the solution changes from blue to green. A further 0·5 ml. of potassium dichromate is added, which changes the colour back to blue. The ferrous sulphate solution is added drop by drop, until the colour of the solution changes from blue to green. This usually occurs fairly critically, and the end-point can be determined to the nearest drop. The total volume of ferrous sulphate solution is noted and recorded to the nearest 0·05 ml. If this volume is x ml., then 1 ml. of the ferrous sulphate solution is equivalent to 10·5/x ml. of the potassium dichromate solution.

5·17 For the estimation of the organic matter content, a small quantity (W) of oven-dried soil is weighed out to the nearest 1 mgm. The size of the sample will vary with the amount of organic matter present, about 5 gm being required for subsoil with a low concentration of organic matter and about 0·2 gm for a very peaty soil. After a number of determinations have been made, experience will indicate the approximate size of sample to be taken. With an unfamiliar type of soil a series of samples of different sizes should be tested and the determination giving a total of 5 to 8 ml. of reduced solution should be taken as giving the correct result.

5·18 The weighed sample is then transferred to a clean 500-ml. conical flask, and exactly 10 ml. of the potassium dichromate solution added from the first burette, followed by 20 ml. concentrated sulphuric acid. The action of the acid on the water liberates heat which promotes the oxidizing reaction, in which the organic matter reacts with the dichromate; to allow this reaction to be completed, the mixture is thoroughly shaken and allowed to stand on a non-conducting surface such as wood or asbestos for about 30 min., during which period the flask must be protected from cold air and draughts.

5·19 After the oxidation, 200 ml. distilled water are added to the mixture followed by 10 ml. phosphoric acid and 1 ml. diphenylamine solution, and the mixture is again shaken. Ferrous sulphate is then added in 0·5-ml. increments until the colour of the solution changes from blue to green, after which an additional 0·5 ml. of dichromate solution is added to restore the colour to blue. Ferrous sulphate is then added drop by drop until the colour of the

solution changes from blue to green after the addition of a single drop. The total volume of ferrous sulphate used is noted and recorded to the nearest 0·05 ml. If this volume is y ml., then the total volume, V, of potassium dichromate used to oxidize the organic matter is given by:—

$$V = 10{\cdot}5\,(1 - y/x)\ (\text{ml.})$$

5·20 The percentage of organic matter present in the oven-dried soil can be calculated from the formula:—

$$\text{Organic matter content} = \frac{0{\cdot}6724\ V}{W}\ (\text{per cent})$$

5·21 This calculation assumes that soil organic matter contains an average of 58 per cent of carbon by weight, and that approximately 77 per cent of the carbon is oxidized by the technique employed.

5·22 If the soil contains chlorides, a correction must be made to the value of the organic matter content by subtracting 1/7th of the chlorine content of the oven-dried soil from the value found. When the chlorine content of the soil is unknown, the effect on the determination can be partly eliminated by dissolving silver sulphate in the sulphuric acid used, to precipitate the chlorine as silver chloride. 20 ml. of a solution containing 1·25 gm silver sulphate in 100 ml. concentrated sulphuric acid should be employed, and will be effective as long as the molecular ratio of chlorine to carbon does not exceed unity.

5·23 The organic matter content is usually reported to the nearest 0·1 per cent based on the original oven-dried weight of the soil.

CARBONATES

5·24 Since carbonates occur in soil chiefly as calcium carbonate or chalk, the carbonate content of a soil is of interest to the engineer because chalky subgrades are susceptible to frost, and also because chalky cohesive soils are more friable and easier to handle than other types of cohesive soil. It is convenient to carry out a carbonate determination in conjunction with a particle-size analysis (see Chapter 3). (The properties of chalk are discussed in more detail in Chapter 7.)

Methods of determining the Carbonate Content

5 25 The most accurate method of determining the carbonate content of a soil is to treat a known quantity with acid, and to measure the amount of carbon dioxide evolved by absorbing it in a standard alkali. A source of error may be the production of carbon dioxide due to the oxidation of organic matter in the soil, e.g. by traces of manganese dioxide. In Schollenberger's method[7] this is counteracted by decomposing the carbonate with hydrochloric acid containing ferrous chloride. The carbon dioxide evolved is absorbed in a standard solution of barium hydroxide, and the excess of alkali is estimated with standard acid. In Hutchinson and MacLennan's method[8] the procedure is similar, except that sodium hydroxide is used as the absorbent and the apparatus required is somewhat simpler.

5·26 For most engineering purposes it is sufficient to make an approximate determination of the carbonate content of a soil. At the Road Research Laboratory, Collins' modification of Schiebler's apparatus[9] has been found to be very useful for routine work. In this apparatus, a weighed amount of soil is treated with hydrochloric acid, and the volume of carbon dioxide given off is measured, corrections being made for temperature and pressure. The apparatus has the advantage that it is compact and simple to operate, and the procedure is rapid enough for about six determinations to be made in an hour.

5·27 In a similar physical method, due to Passon, the pressure exerted by the gas evolved is measured by a manometer calibrated by using known quantities of calcium carbonate.

Determination of the Carbonate Content of Soil: Collins' Method

5·28 The apparatus and reagents required are as follows:—

(1) Thermostatically controlled drying oven, set to 105 - 110°C.
(2) Analytical balance, accurate to 0·001 gm.
(3) The apparatus shown in Plate 5·1B.
(4) Hydrochloric acid solution (one volume of concentrated acid diluted with three volumes of distilled water).

5·29 The apparatus consists of a large glass jar, in which the various working parts are immersed in water to maintain a uniform temperature throughout the experiment. The water is stirred by blowing air bubbles through it from the rubber hand-pressure bulb provided. Carbon dioxide is generated in the small conical flask which is immersed in the water bath during the experiment and is connected to a measuring tube graduated from 0 to 50 ml. in steps of 0·1 ml. The volume of gas produced in the flask is determined by noting the fall in the level of water in this tube, after the pressure in the flask has been adjusted to that of the surrounding atmosphere.

5·30 The procedure for the determination is as follows:—A sample of oven-dried soil between 0·1 gm and 20 gm is weighed out accurately to the nearest 1 mgm (W_1). The actual quantity taken is dependent on the carbonate content of the soil, and it is often convenient to carry out several preliminary determinations, starting with a small quantity, and increasing the amount of soil taken, until a reading of about 10 to 25 ml. of gas evolved is obtained.

5·31 The weighed sample is carefully transferred to the conical flask, and 10 ml. of hydrochloric acid are added to the small graduated tube contained in the flask; 15 ml. of hydrochloric acid may be required for soils containing a high proportion of carbonates. The tube is placed in the conical flask, care being taken not to spill any of the acid on to the soil, and the flask is connected to the apparatus by inserting the rubber bung at the end of the connecting tube. The flask is then placed in the water jacket, where it is held under the surface by a small wire hook on the connecting tube which can engage on a loop on the protective wire screen. The water in the jar is stirred by closing the right-hand vertical tap (see Plate 5·1B), and blowing in air from the pressure bulb.

5·32 Both taps are then opened, and the water level in the volume-measuring tube is brought up to the zero mark by gentle pressure from the bulb. The

water level in the adjacent tube should automatically rise to the same level. The left-hand horizontal tap is then closed, and the pressure on the bulb is released, allowing the water level in the right-hand tube to sink some distance. The water level in the measuring tube will also sink to some extent when this is done.

5·33 The conical flask, still connected to the measuring tube, is removed from the jacket and the acid is allowed to run over the soil by tilting the flask. The latter is again inserted in the jar, and the water is agitated with air from the pressure bulb after the right-hand vertical tap has been closed. Evolution of gas causes a depression of the water level in the measuring tube and when the reaction is seen to be complete, the water level in the adjacent tube is raised by means of the pressure bulb until it is level with that in the measuring tube. The process of shaking the flask, agitating the water jacket and adjusting the water level is repeated until no further increase in the volume of gas evolved is observed. The increase in the volume of gas in the system is then noted from the measuring tube, and recorded.

5·34 The temperature of the water jacket before and after the reaction should be observed by means of the thermometer which is suspended in the jar, and the two values obtained, as well as the average value, should be recorded. One tenth of a millilitre should be subtracted from the measured increase in the gas volume for every 0·2°C. rise in temperature, or a similar volume added for a corresponding decrease, to allow foı expansion or contraction of the gas. The value of the barometric pressure during the experiment should also be noted and recorded.

5·35 The weight in milligrams of carbon dioxide or calcium carbonate in the soil sample can be conveniently calculated with the slide rule provided with the apparatus (Plate 5·2A). The figure corresponding to the average temperature is set opposite the figure corresponding to the barometric pressure, and the weight in milligrams of CO_2 or $CaCO_3(W_2)$ is then given by the figure opposite the volume in ml. of acid used.

5·36 The percentage of carbon dioxide or calcium carbonate in the sample is then calculated as a percentage of the total weight, i.e.

$$CO_2 \text{ or } CaCO_3 = \frac{W_2 \times 100}{W_1} \text{ (per cent)}$$

5·37 It is usually sufficient to report the carbon dioxide or calcium carbonate content in a soil to the nearest 1 per cent by weight of the original oven-dried soil.

SULPHATES

5·38 The soluble sulphates usually found in soil are those of Ca, Na and Mg. They can cause disintegration of concrete road slabs and pipes, and corrosion of metal drainage pipes in contact with the soil, and in arid areas they can also disrupt the structure of the soil on an earth road by forming " salt boils." A more detailed discussion of the occurrence and properties of soils containing soluble sulphates is given in Chapter 2.

Methods of determining the Sulphate Content

5·39 When determining the sulphate content of a soil it is usually satisfactory to prepare an aqueous extract of the soil, since all the salts concerned are sufficiently water-soluble for the purpose. The sulphate content of this extract or an aliquot portion of it is then determined in the normal manner. This can be done gravimetrically, by precipitating the sulphate as barium sulphate, filtering off the precipitate and weighing it. Alternatively, volumetric methods have been proposed which appear to be more rapid[10] [11]. These include the estimation, by titration with potassium permanganate, of the amount of precipitate formed when the sulphate is precipitated as benzidine sulphate. Another volumetric method consists of precipitating the sulphate with a known excess of barium chromate in acid solution, followed by a determination of the excess soluble chromate by titration with thiosulphate, after removal of unreacted barium chromate by alkalization.

5·40 At the Road Research Laboratory, the soluble salts are extracted from the soil with water, the sulphates precipitated with barium chloride and the insoluble barium sulphate filtered off and weighed. This method has the advantages that it is relatively simple to carry out and is fairly accurate, while the apparatus required is simple.

Determination of the Sulphate Content

5·41 The apparatus and reagents required are as follows:—

(1) Bottle shaker of the type indicated in Plate 5·2B with bottles of 300-ml. capacity, fitted with rubber bungs, or any other suitable apparatus.
(2) Electric muffle furnace, or Méker burner.
(3) Analytical balance, accurate to 0·001 gm.
(4) Porcelain crucibles 40 mm. in diameter.
(5) 500-ml. beakers, filter funnels, filter papers, glass rods, etc.
(6) 5 per cent W/V solution of barium chloride (A.R. grade).

5·42 About 10 gm of oven-dried soil are weighed accurately to the nearest 1 mgm (W_1). These are transferred to the shaking bottle, about 150 ml. of distilled water are added and the rubber bung is inserted. The bottle is then placed in the shaker, and the contents shaken for 30 min.

5·43 After shaking, the soil suspension is filtered and the filtrate collected in a 500-ml. beaker. The soil on the filter paper should be washed with a further 50 ml. of distilled water. The extract is acidified with a few drops of hydrochloric acid and then brought to boiling point over a bunsen flame, and barium chloride solution is added slowly to the hot liquid until no further precipitate is obtained when drops of the solution are added.

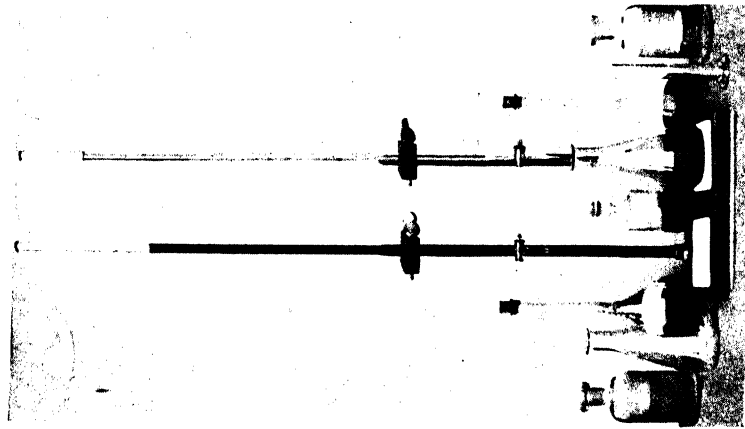
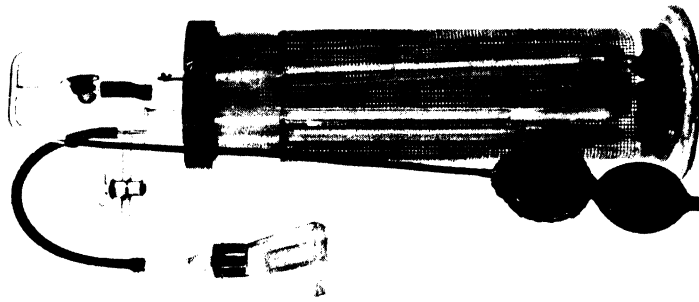
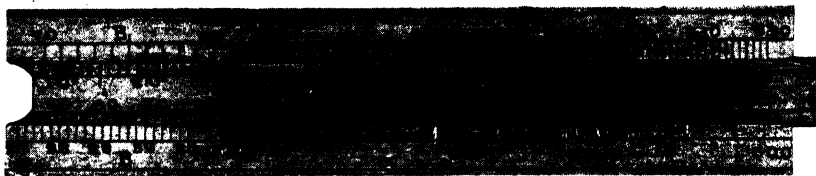
5·44 The precipitate is filtered on an ashless filter paper (Whatman No. 44), and washed with hot distilled water until the washings do not give a cloudy solution when added to a small quantity of silver nitrate solution. The moist filter paper is then folded round the precipitate and placed in the crucible, which should previously have been ignited to constant weight to an accuracy of 1 mgm. The crucible is placed on a pipe-clay triangle standing on a tripod and warmed gently until the filter paper chars. Care must be taken during

(A) APPARATUS FOR DETERMINING THE ORGANIC MATTER CONTENT OF SOIL (dichromatic oxidation method)

(B) COLLINS' APPARATUS FOR DETERMINING THE CARBONATE CONTENT OF SOIL

PLATE 5·1

(A) SLIDE-RULE FOR USE WITH COLLINS' APPARATUS
for the determination of carbonates in soil

(B) MECHANICAL SHAKER FOR PREPARATION OF SOIL EXTRACTS
during the determination of soluble sulphates in soil

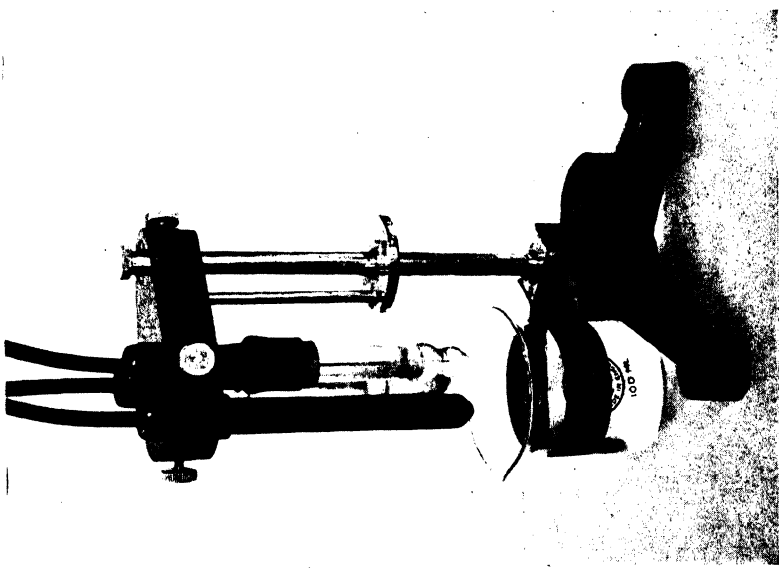



PLATE 5·2

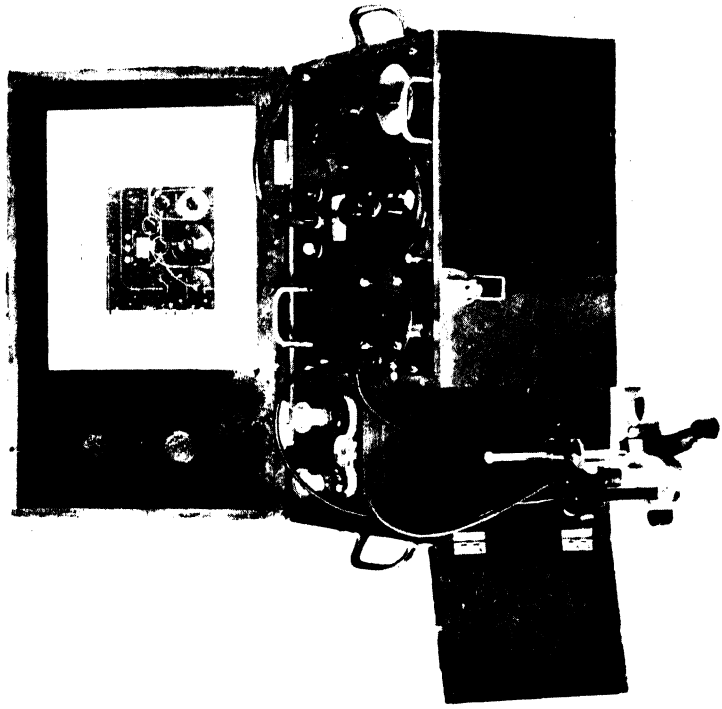
(A) ELECTRODE ASSEMBLY FOR pH MEASUREMENT
with temperature compensator

(B) ELECTROMETRIC APPARATUS FOR pH
DETERMINATION

PLATE 5·3

APPARATUS FOR THE COLORIMETRIC DETERMINATION OF SOIL pH

PLATE 5·4

this operation not to allow the paper to inflame, as some of the precipitate may then be lost in the resulting hot air currents. When the paper has been completely charred, the crucible is raised to red heat in the electrical muffle furnace for about one hour to burn off the carbon. Alternatively, this may be done by heating on the pipe-clay triangle over a Méker burner.

5·45 The crucible is then removed to a desiccator and cooled, and when cold it is weighed accurately to the nearest 1 mgm. The difference between this weight and that of the empty crucible previously determined gives the weight of the barium sulphate precipitate (W_2) in grams.

5·46 The sulphate content is usually expressed in terms of the percentage of sulphur trioxide ($-SO_3$) in the original oven-dried sample. This is given by the formula:—

$$SO_3 = \frac{34{\cdot}3 \times W_2}{W_1} \text{ (per cent)}$$

5·47 It is usually sufficient to report the sulphur trioxide content in a soil to the nearest 0·01 per cent by weight of the original oven-dried soil.

SOIL REACTION

5·48 The hydrogen ion concentration or pH of the soil water is of interest in engineering problems connected with the corrosion of metals in contact with the soil and in soil stabilization processes using resinous materials. The significance of the soil pH is discussed in Chapter 2 and its influence in resin stabilization is referred to in Chapter 14.

Methods of determining Soil pH

5·49 The hydrogen ion concentration in soil water may be determined either electrometrically or colorimetrically. The colorimetric methods are generally simpler to carry out and best suited for rapid field work, whereas the methods involving electrometric measurements are more accurate, but the apparatus required is of greater complexity.

5·50 At the Road Research Laboratory, a small testing kit based on Kühn's method[12] is employed for field work. A small quantity of soil suspension is shaken up with a solution of a universal indicator, and the resulting colour is compared with a chart giving the colours obtained at various pH values. For more accurate laboratory work, the potential developed between two special electrodes (Plate 5·3A) immersed in a soil suspension is determined with a sensitive millivoltmeter which has been suitably calibrated in pH units (Plate 5·3B). This instrument will read to 0·02 of a pH unit, while the colorimetric method previously referred to can be used to an accuracy of 0·25 of a pH unit.

5·51 A description of the colorimetric method only is given below, since it is felt that this method is sufficiently accurate for most soil engineering purposes at the present time. If it is necessary to carry out more accurate measurements, however, the electrometric apparatus should be employed, and a suitable textbook or references consulted. The makers of these instruments supply instruction books which give the detailed procedure required.

Determination of the Soil pH (Colorimetric Method)

5·52 The apparatus (see Plate 5·4) and reagents required are as follows:—

(1) Test tubes, 8 in. long and ½ in. in diam., suitably graduated.
(2) B.D.H. barium sulphate for soil-testing.
(3) B.D.H. soil indicator.
(4) Colour comparison chart.
(5) Distilled water.

5·53 A small quantity of the soil to be tested is added to the test tubes, followed by a quantity of the barium sulphate powder. The relative amounts of the two materials required will differ according to the type of soil, but the following values indicating the thickness of each layer in the test tube will serve as a general guide:—

Sandy soils	— ½ in.	barium	sulphate,	1½ in. soil
Silty soils	— 1 in.	„	„	1 in. soil
Clay soils	— 1½ in.	„	„	½ in. soil

5·54 Distilled water is then added until the level is at the lower of the two graduations on the test tube, followed by sufficient of the soil indicator to bring the level up to the second mark. A small rubber bung is then placed in the mouth of the tube, which is shaken vigorously to bring the soil into suspension, after which the tube and contents are allowed to stand for some minutes.

5·55 During this period, the coarser soil particles will settle to the bottom of the tube, and the finer particles will be flocculated by the particles of barium sulphate, leaving a relatively clear coloured layer of liquid. The colour of this liquid is then compared with the coloured panels on the chart, in good daylight, by holding the section of the tube containing the clear liquid against the various panels in turn, and an estimate is thus made of the pH value of the solution.

5·56 It is usually sufficient to report the pH of the soil to the nearest 0·25 of a pH unit when this method is used.

SUMMARY

5·57 This chapter describes some of the chemical characteristics of soil that are of interest to the road engineer, and indicates how they may affect the behaviour of soil in practice. The particular characteristics dealt with are the organic matter content, the carbonate and sulphate contents, and the soil reaction or pH value.

5·58 The methods available for the laboratory determination of these constituents or properties are indicated very briefly, and descriptions are given of the procedures at the Road Research Laboratory that are considered suitable for use in laboratories engaged in testing soils for engineering purposes.

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CHAPTER 6

ROADMAKING AGGREGATES

INTRODUCTION

6·1 Aggregate is the basic material of road construction. It forms the greater part of the body of the road; it is called upon to bear the main stresses occurring in the road and to resist wear from surface abrasion. The properties of aggregates are therefore of considerable importance to the road engineer, and some knowledge of the characteristics of different types of aggregate is helpful in the design of road structures. This chapter sets out briefly the main facts of the geology, production and testing of aggregates, with particular reference to the variability of samples from the same or different sources.

DEFINITION

6·2 The British Standard Glossary of Highway Engineering Terms (B.S. 892:1940) defines aggregate as " the inert mineral fragments and particles forming the main structure of a mixture such as asphalt, tarmacadam or concrete," and distinguishes between coarse and fine aggregates. No dividing line is given between the coarse and fine aggregates, as this varies according to the purpose for which they are to be used: for example, in concrete aggregates the division is made on the $\frac{3}{16}$-in. square mesh sieve, while for bituminous aggregates it is more often $\frac{1}{8}$-in. For definitions of the numerous other terms used in the description of aggregates, the Glossary should be consulted.

AGGREGATE TYPES: GEOLOGY AND PETROLOGY

6·3 The raw material from which most road aggregates are prepared is natural rock. There are many different types of rock, but all are composed of grains or crystals of minerals held together in a variety of ways. The properties of a rock therefore depend on the properties of its constituent minerals and the nature of the bond between them, i.e. on the composition, grain size and texture of the rock, factors which depend to a large degree on its mode of origin. Natural rocks fall into three well defined groups, according to their mode of origin—igneous, sedimentary and metamorphic—and rocks of any of these groups may occur either massive or as gravel. The only artificial road aggregate of any importance is slag from blast or steel furnaces.

Igneous Rocks

6·4 These are formed by the cooling and crystallization of molten rock that has been injected into the earth's crust from below, and may be regarded as the ultimate origin of all the rocks forming the solid crust of the earth. They are estimated to form about 95 per cent by volume of the outer ten miles of the earth's crust. At the surface, however, they are extensively covered by sedimentary rocks.

6·5 The composition, grain size and texture of igneous rocks vary considerably according to the conditions under which the molten rock cooled. Table 6·1 shows how the main groups of igneous rock are related; it gives examples of rock types in each group and shows how some of the properties of the rocks vary. It should be noted that the variation in properties, like the rock types themselves, shows a continuous gradation, each rock type merging imperceptibly into its neighbours. It is also necessary to remember that only general trends in the variation of properties can be indicated. There are numerous exceptions to these general trends, and the properties of any one sample cannot be designated by mere reference to the rock type, but must be determined by applying the appropriate tests, which are dealt with in a later section of this chapter. No precise reference to mechanical strength is made here, because mechanical strength varies widely in each group; this subject is dealt with more fully in the section on the applications of test results.

6·6 An additional group of rocks—the ultrabasic—is not included in Table 6·1 because such rocks are of comparatively little importance as roadstones. They are characterized by a very low silica content, dark colour and high specific gravity.

6·7 Igneous rocks vary not only in composition and grain size but also in texture. The constituent crystals may fit together in a kind of mosaic, or they may be intergrown (ophitic texture in basic rocks, granophyric texture in intermediate and acid rocks), or relatively large crystals of one mineral may be set in a matrix or groundmass of small crystals of the same or other minerals (porphyritic texture). Coarse grain is generally undesirable in roadstones as it leads to brittleness and crushing under the roller. Extremely fine-grained rock is also undesirable because when crushed it is liable to form a harsh, sharp-edged aggregate. Any kind of intergrowth of the constituent minerals is highly desirable because it adds greatly to the strength of the rock. Many igneous rocks of medium grain size have an intergrown texture and are amongst the best of roadstones.

6·8 The term " texture," as applied to the body of the rock, should be clearly distinguished from surface texture, which applies to the surface only, and is an important factor affecting the adhesion of binders to the rock. Although surface texture is to some extent dependent on the texture of the body of the rock, it is considerably affected by the nature of the rock fracture and by the amount of abrasion the surface has suffered, especially in the case of gravels.

6·9 All igneous rocks are subject to decomposition by weathering on exposed surfaces or along fissures, and internally by the chemical instability of their minerals. In general, these factors require time on a geological scale to operate to any extent. A rock that is reasonably sound and fresh when quarried is unlikely to suffer from decomposition during the life of the road into which it is built, although a partially decomposed rock may. It is easy to recognize badly decomposed rock by patchy discolouration and by its soft and friable nature: such rock should not be used as roadstone. Petrological analysis will sometimes show that an apparently sound rock is partially decomposed and will be likely to suffer further decomposition during the life of the road; its use should be avoided if possible.

TABLE 6·1

CLASSIFICATION AND PROPERTIES OF IGNEOUS ROCKS

ACID (Over 66% total SiO_2)*	INTERMEDIATE (55-66% total SiO_2)*	BASIC (Under 55% total SiO_2)*
COARSE-GRAINED (" PLUTONIC ") ROCKS. Grain size larger than about 1/20 in. Liable to be brittle owing to presence of large crystals. The coarsest-grained of these rocks are unsuitable for roadstone. Examples of rock types:—		
Granite Granodiorite (Widely distributed in the British Isles)	Syenite Diorite (Comparatively rare in the British Isles)	Gabbro Norite (Not very common in the British Isles)
MEDIUM-GRAINED (" HYPABYSSAL ") ROCKS. Grain size between about 1/200 and 1/20 in. Very frequently possess intergrown texture: include some of the best roadstones. Examples of rock types:—		
Microgranite Granophyre (Fairly common in the British Isles)	Porphyry Porphyrite (Fairly common in the British Isles)	Dolerite Diabase (Widely distributed in the British Isles)
FINE-GRAINED (" VOLCANIC ") ROCKS. Grain size below about 1/200 in., i.e. below the limit of visible recognition. Similar to medium-grained rocks, but sometimes liable to be brittle and splintery. Examples of rock types:—		
Rhyolite Felsite (Not very common in the British Isles)	Trachyte Andesite (Not very common in the British Isles)	Basalt Spilite (Widely distributed in the British Isles)

← Continuous variation in properties →

Light colour → Dark colour
(Due to increase in ferromagnesian minerals)

Low specific gravity → High specific gravity
(2·6) (Due to increase in ferromagnesian minerals) (2·9)

{Chemical stability generally decreases with decreasing silica content, but no sound rock is likely to decompose during the life of a road.} →

{A general tendency towards better adhesion to bituminous binders has been noted in basic rocks, but the subject is not well understood yet, and good and bad adhesion have been obtained with all types of rocks.} →

*These figures refer to SiO_2 (silica), much of which is in combination with alumina, etc., to form feldspars and other minerals. Only acid rocks contain any appreciable amount of free SiO_2 as quartz.

Sedimentary Rocks

6·10 The exposed surfaces of rocks are subject to the continuous attack of weathering agents, which break them down by mechanical disintegration and chemical decomposition.

6·11 The larger pieces of rock—products mainly of mechanical disintegration—form gravels, and the smaller—products of weathering—are carried away, mainly by running water, and deposited elsewhere to form the sedimentary rocks. Vast thicknesses of such deposits have accumulated over geological time.

6·12 The minerals produced by decomposition are much simpler than those of the original igneous rock. They can be divided into three main classes—calcareous, siliceous (sometimes called arenaceous) and argillaceous—which give rise to three corresponding classes of sedimentary rocks.

6·13 The calcareous minerals are soluble in water and are carried away in solution. Small marine animals extract them from the solution, and the remains of these animals are deposited on the sea floors in great thicknesses, to appear later in geological time as chalk or limestone.

6·14 Insoluble siliceous minerals form sand and silt, which may remain loose and incoherent, or may be lithified by the pressure of overlying strata or by the deposition of cementing material between the grains. The resulting rock is either a sandstone, greywacke (sandstone with a proportion of fine volcanic ash), arkose (sandstone with a proportion of the mineral feldspar), siltstone, or quartzite (sandstone with siliceous cement). Soluble siliceous minerals are removed in solution and deposited as flint and chert in a way similar to that described above for limestone and chalk, with which they are almost invariably associated.

6·15 The argillaceous minerals are very fine-grained but insoluble. They are deposited as muds and clays, which under pressure from overlying deposits become mudstones and shales.

6·16 In addition to these three main classes of sedimentary rocks, mixtures such as siliceous or argillaceous limestones, calcareous sandstones and so on, frequently occur.

6·17 Sedimentary rocks are of much simpler mineral composition than igneous rocks, and show less variation in grain size and texture. Their distinguishing characteristic is that, having been deposited in layers, they have a stratified or laminated structure. In geologically young rocks this may be a source of mechanical weakness, but in the older rocks the planes of stratification although still visible are no longer planes of weakness, having been welded by age-long pressure. Table 6·2 shows the main rock types occurring in the three classes of sedimentary rocks, with particular reference to their suitability as roadstone. As with Table 6·1 for igneous rocks, only general trends are indicated, the properties of rocks of each type showing wide variations in different samples.

Metamorphic Rocks

6·18 Shrinkage and buckling of the earth's crust periodically subjects the rocks to enormous pressures over large areas and for long periods of time The movements of these rocks may force up masses of molten rock from below into the upper crust and on to the surface. The solid rocks of the crust are considerably modified in mineral structure by the great heat and pressure developed; rocks so altered are known as metamorphic rocks.

TABLE 6·2

CLASSIFICATION AND PROPERTIES OF SEDIMENTARY ROCKS

CALCAREOUS (Predominant mineral: calcite, $CaCO_3$)	
LIMESTONE (including calcite-mudstone)	Softer than sound igneous rocks, but the best limestones (mainly of Carboniferous age) appear to have adequate strength for most road-making purposes. Usually light in colour. Specific gravity* 2·65—2·75. Generally have excellent adhesion to bituminous binders. Widely distributed in the British Isles.
DOLOMITE (including dolomitic limestone)	Limestone in which part or all of the calcite has been replaced by dolomite ($CaMg.(CO_3)_2$). General properties as limestone, but slightly stronger, and higher specific gravity* (2·70—2·80). Fairly common in the British Isles.
CHALK	Too soft for use as road aggregate.
SILICEOUS (Predominant mineral: quartz or chalcedony, both SiO_2)	
SANDSTONE (including siltstone, greywacke and arkose)	Liable to have a laminated structure, but if older than Carboniferous may be as strong as igneous rocks. Variable colours—red, blue, green, grey, etc. Specific gravity* 2·60—2·75. Adhesion to bituminous binders probably not quite so good as limestone. Types suitable for roadstone less widely distributed than limestone.
QUARTZITE	Very hard but inclined to be brittle. Adhesion to bituminous binders variable, but inclined to be rather poor. Colour usually light. Specific gravity* 2·55—2·65. Fairly common in the British Isles, especially as gravel.
FLINT	See section on gravels.
CHERT	Very hard and liable to be brittle. Not often used for roads.
ARGILLACEOUS (Clay minerals predominate)	
CLAY / SHALE	Very fine-grained, soft and often laminated. Unsuitable for use as road aggregates.
MUDSTONE	Very fine-grained, often laminated and with splintery fracture. Some varieties useful as roadstone, but not widely distributed in the British Isles.

*The values given for specific gravity refer to rocks suitable for use as roadstone: geologically young and porous rocks have much lower specific gravities.

6·19 Rocks altered by heat alone, without any considerable pressure, are called "thermal metamorphic rocks," and the process of thermal metamorphism almost invariably results in a rock that is harder and tougher than the original. Rocks of simple composition merely undergo recrystallization—sandstones are converted to quartzites and limestone to marble. Rocks of more complex composition, such as igneous rocks and impure sedimentary rocks, frequently suffer such considerable change in mineralogical structure that the nature of the original rock is completely obliterated, the resulting rock

being known as hornfels. These rocks often show the interlocking of minerals that is such a useful asset in roadstone. From the roadmaking point of view, therefore, rocks are nearly always improved by thermal metamorphism; some hornfels are among the best of roadstones, although they are not widely distributed in this country.

6·20 Rocks altered by pressure alone (dynamic metamorphism) are rare, as pressure is almost invariably accompanied by heat, giving rise to the " regional metamorphic rocks," the main characteristic of which is a banded or laminated structure. In gneiss and granulite the bands are of irregular occurrence and widely spaced; these rocks resemble coarse-grained granite in their roadmaking properties and are liable to crush under the roller. In slate and schist the laminations are closely spaced, resulting in a highly fissile rock unsuitable for roadstone. Some rocks referred to as hornblende-schists, however, resemble dolerites and basalts in their roadmaking properties, but these rocks are not typical schists.

Gravel

6·21 The coarser material resulting from the disintegration of natural rocks (particles of sizes ranging from about ¼ in. to 2 in.) is carried away by rivers and deposited as gravel. During transit the particles are worn down by attrition and become more or less rounded in shape, with smooth surfaces.

6·22 A gravel may consist almost entirely of one type of hard rock, such as flint or quartzite, or may contain a wide variety of rocks of different type and hardness. Gravels consisting entirely of soft rock are uncommon, although softish limestone gravels occur and have been used extensively for road construction in districts that are distant from sources of more suitable aggregate. The distinguishing feature of gravel is therefore not rock type, but that it always consists of more or less rounded or irregular particles with relatively smooth surfaces.

6·23 The most important gravel deposits in this country are in the east and south-east, where flint gravels are the main source of local aggregates, and in the Midlands, where the quartzite gravels of the Trent valley are used extensively for roads. Both are very hard and inclined to be brittle—especially the flint—but on the whole they are good road aggregates. Softish limestone gravels, occurring in parts of Oxfordshire, Gloucestershire and elsewhere have been used extensively for roads and are found to be satisfactory aggregates for concrete.

6·24 Mixed gravels occur in widely scattered districts in the North of England and in Scotland; often one deposit contains a variety of both igneous and sedimentary rocks. In general their properties depend on the properties of the constituent rocks and the proportions in which they are present. Mixed gravels are liable to contain a variable proportion of soft particles—chalk, clay lumps, shale or soft sandstone—which if present in a high proportion are a source of weakness

Sand

6·25 Natural sands consist largely of the final residue of resistant mineral grains resulting from rock-weathering, and have often been through many

cycles of deposition and weathering. The most abundant mineral in sands is quartz, since, unlike most other common rock-forming minerals, this substance is hardly affected by any of the ordinary weathering agents. Where sand occurs in association with gravel, however, the larger sand particles are mainly chips from the gravel; thus the sands occurring with flint gravels are mainly quartz, but the larger sand particles are flint.

6·26 Apart from natural sand, the fines from crushed rock are sometimes used as fine aggregate for road construction. These " crushed stone sands " naturally have properties resembling those of the parent rock, and call for little comment except that the particles comprising them are liable to be splintery and flaky in shape.

Slag

6·27 Blast-furnace slag is used extensively as road aggregate. It is composed of a number of alumino-silicates of calcium and magnesium, minerals that are found in some basic igneous rocks. Slag is a much more variable material than natural rock owing to its rapid cooling which results in greater differences in grain-size and porosity between the interior and exterior of the cooling masses, the exterior often cooling so rapidly as to form an amorphous glass.

6·28 Slag is sometimes chemically unstable, owing to residual sulphur and iron, and the possible presence of calcium orthosilicate in a form that is liable to undergo volume-change at normal temperatures.

6·29 Nevertheless, if manufactured under carefully controlled conditions slag makes an excellent road aggregate. The requirements for slag intended for this purpose are described below in the section on testing.

Trade Groups of Roadmaking Aggregates

6·30 Petrologists have identified and named many hundreds of different types of natural rock. It is not necessary, however, for the road engineer to know the names of all these types, many of which do not differ from each other in their roadmaking characteristics. The British Standard nomenclature for roadstone (B.S. 812) recognizes 11 " Trade Groups " of roadmaking aggregates, each group containing rocks that are similar to one another from the roadmaking point of view, i.e. in composition, grain-size and texture.

6·31 Table 6·3 gives a list of the trade groups, with examples of the types of rock occurring in each group. The key to Table 6·3, used in conjunction with Tables 6·1 and 6·2, gives a more precise idea of the petrological classes into which fall the different types of rock in each trade group.

6·32 Terms that are of local significance only, such as " whinstone," " elvan," " toadstone," " pennant stone," are not included in this scheme because they do not refer to specific rock types, and are frequently misused. In British Standard 812 it is recommended that roadmaking rocks be referred to by their trade group names.

TABLE 6·3

TRADE GROUPS OF ROADMAKING AGGREGATES

Rock	Classification
"ARTIFICIAL" GROUP	
Slag	
BASALT GROUP	
Andesite	Ig. Int. F.
Basalt	Ig. Bas. F.
Basic porphyrites	Ig. Int. M.
Diabase	Ig. Bas. M.
Dolerite	Ig. Bas. M.
Epidiorite	Met. R.
Hornblende-schist	Met. R.
Lamprophyre	Ig. Bas.
Quartz-dolerite	Ig. Bas. M.
Spilite	Ig. Bas. F.
Teschenite	Ig. Bas.
Theralite	Ig. Bas.
FLINT GROUP	
Chert	Sed. Si.
Flint	Sed. Si.
GABBRO GROUP	
Basic diorite	Ig. Int. C.
Basic gneiss	Met. R.
Gabbro	Ig. Bas. C.
Hornblende rock	Ig. U.
Norite	Ig. Bas. C.
Peridotite	Ig. U.
Picrite	Ig. U.
Serpentine	—
GRANITE GROUP	
Gneiss	Met. R.
Granite	Ig. Ac. C.
Granodiorite	Ig. Ac. C.
Granulite	Met. R.
Pegmatite	—
Quartz-diorite	Ig. Int. C.
Syenite	Ig. Int. C.
GRITSTONE GROUP	
Agglomerate	Sed.
Arkose	Sed. Si.
Breccia	Sed.
Conglomerate	Sed.
Greywacke	Sed. Si.
Grit	Sed. Si.
Sandstone	Sed. Si.
Tuff	Sed.
HORNFELS GROUP	
Contact-altered rocks of all kinds except marble	Met. Th.
LIMESTONE GROUP	
Dolomite	Sed. Ca.
Limestone	Sed. Ca.
Marble	Met. Th.
PORPHYRY GROUP	
Aplite	—
Dacite	Ig. Ac. F.
Felsite	Ig. Ac. F.
Granophyre	Ig. Ac. M.
Keratophyre	Ig. Int. F.
Microgranite	Ig. Ac. M.
Porphyry	Ig. Int. M.
Quartz-porphyrite	Ig. Int. M.
Rhyolite	Ig. Ac. F.
Trachyte	Ig. Int. F.
QUARTZITE GROUP	
Ganister	—
Quartzitic sandstone	Sed. Si.
Recrystallized quartzite	Met. Th.
SCHIST GROUP	
Phyllite	Met. R.
Schist	Met. R.
Slate	Met. R.

KEY

Ig. = Igneous
Sed. = Sedimentary
Met. = Metamorphic

Ac. = Acid
Int. = Intermediate
Bas. = Basic
U. = Ultrabasic

C. = Coarse-grained
M. = Medium-grained
F. = Fine-grained

Ca. = Calcareous
Si. = Siliceous

Th. = Thermal
R. = Regional

NOTE. It is recognised that traditional names are in use for describing certain rocks. If such terms are used, the rock in question should also be described by the appropriate trade group name from the list above.

Petrological Causes of Variability in Aggregates

6·33 These are many, and often of a complex character. Only a few examples of the more significant causes can be given here. In igneous rocks, different rates of cooling in different parts of the rock mass result in variability of grain size and texture. In addition, segregation of minerals while the rock was still fluid has often caused variability in the composition of a rock. Differences between successive strata are the commonest cause of variability in sedimentary rocks. Changing conditions at the time of deposition may result in beds of shale or clay alternating with those of limestone or sandstone. Succcessive beds may show varying degrees of consolidation or cementation. Chemical changes—such as silicification and dolomitization—brought about by solutions circulating through fissures and pores, often produce local variations in the rock. The extent to which a metamorphic rock is altered varies with its distance from the seat of metamorphism, especially in the case of thermal metamorphism. This variability may be superimposed on all the other types of variability found in igneous and sedimentary rocks, so that metamorphic rocks tend to be more variable than others.

6·34 It will thus be seen that the mode of formation of natural rocks causes them to be highly variable materials. For a rock to be uniform over any considerable area is the exception rather than the rule. In most quarries uniformity of the products can only be maintained by careful selection of the rock quarried, and relaxation in this respect is a common cause of unsuitable aggregates.

6·35 Gravels sometimes show variability in the proportion of soft or otherwise unsuitable particles that they contain, but this problem is rarely serious in this country, where the most important roadmaking gravels have a satisfactory degree of uniformity. The silt, clay and organic content of gravels and sands is a variable factor which should be reduced to harmless proportions.

6·36 Slag, as mentioned in para 6·28, is liable to chemical instability in addition to variability in grain size and texture; the latter may be glassy or crystalline, dense or honeycombed, according to the conditions under which it was cooled.

6·37 A type of variability which is often incorrectly attributed to the petrological characteristics of rocks is variability in the particle shapes of aggregates, which sometimes contain an excessive proportion of undesirably flaky and elongated particles. Although some rocks do show a tendency to crush into flaky particles, the real cause of poor aggregate shape most often lies in the crushing process; this is dealt with below.

AGGREGATE PRODUCTION

Crushed Stone Quarries

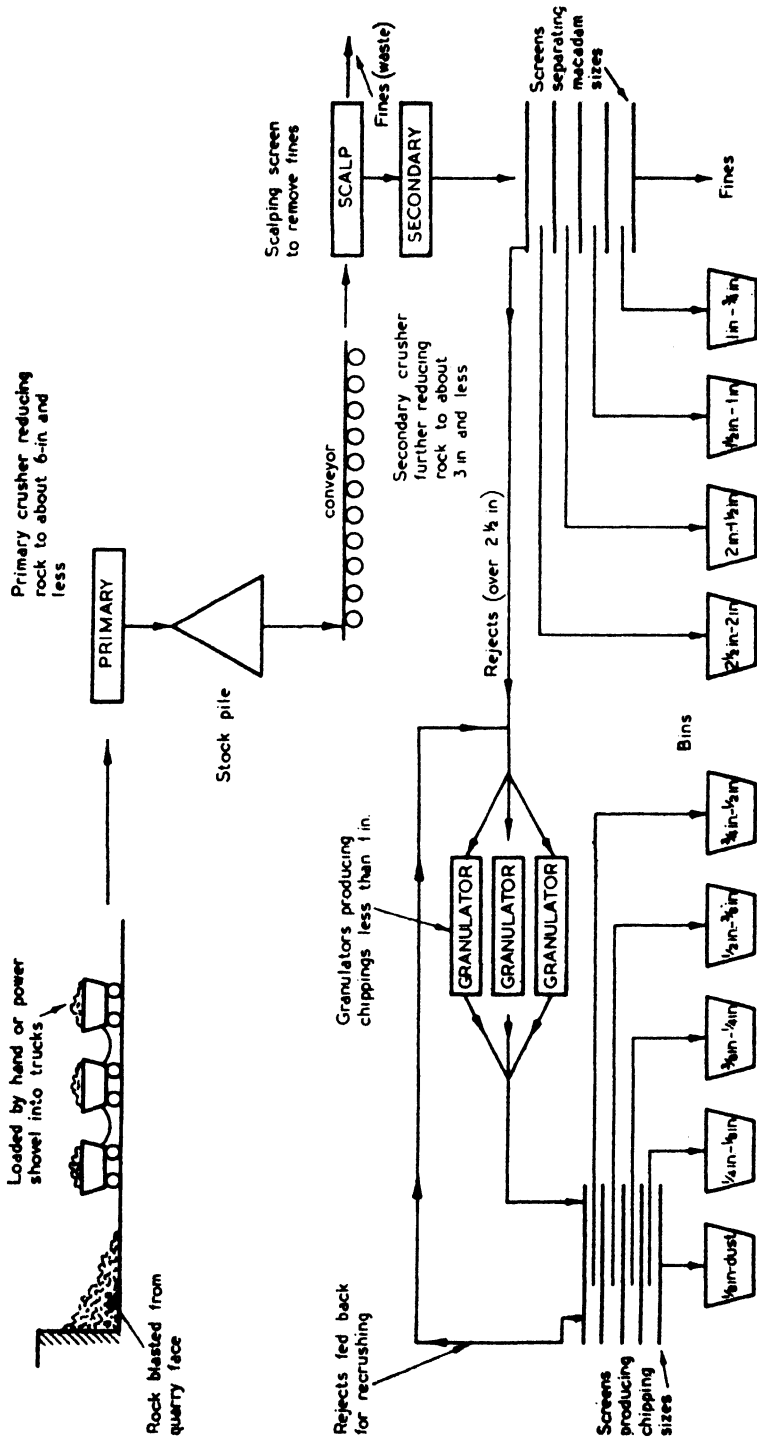
6·38 The essential processes of quarrying for crushed rock and of producing road aggregates are illustrated in Fig. 6·1 and briefly discussed below, as they enable a better understanding to be obtained of the causes of poor or variable aggregates that arise in the production processes. The actual flow-sheets of quarries differ widely according to the type of rock quarried, the size of the

Note: This diagram is intended to illustrate the processes involved; it does not represent the flow sheet of any one quarry. The actual arrangement used varies considerably from one quarry to another.

FIG. 6·1 SEQUENCE OF PROCESSES IN MANUFACTURE OF ROAD AGGREGATES

quarry and many other factors. The old-fashioned quarry is a primitive affair with a crusher and screens; the modern roadstone quarry is a complex unit with elaborate plant backed by an efficient maintenance shop.

6·39 QUARRYING OPERATIONS. The overburden of soil and unsuitable top rock have to be cleared at least 20 to 30 ft back from the working face, otherwise the useful rock may be contaminated.

6·40 The practices employed for drilling and blasting the rock vary considerably, and do not greatly affect the quality of the product. In large-scale blasting 10,000 to 20,000 tons of rock may be brought down at each blast.

6·41 Pieces of rock that are too large for the crushers are reduced by secondary blasting, which may be done either by drilling and charging the holes (" pop-shooting ") or by placing the charge on the surface of the rock and covering it with a plaster of clay (" plaster-shooting ").

6·42 In some quarries the rock is loaded into trucks or dumpers by hand; this method is useful when it is necessary to sort out unsuitable material. More often, however, loading is by mechanical shovels of sizes up to 4½ cu. yd; ¾ and 1 cu. yd are popular sizes and shovels of these sizes will load about 25 and 40 tons/hour respectively. The shovels load into dumpers, tipping wagons or railway trucks of 3½ to 15 tons capacity for transport to the crushing plant.

6·43 Mechanical loading means that everything that is brought down by the blast is sent to the crushers, unless some form of hand-sorting is employed; it is, therefore, most suited to clean, uniform rock, and there can be little doubt that in many quarries the introduction of mechanical loading has resulted in a deterioration in the quality of the aggregates produced, owing to the admixture of unsuitable material.

6·44 CRUSHING. To produce high-quality road aggregates, it is necessary to employ at least two stages of crushing in order to reduce the rock from the quarry face to suitable sizes. In this country the primary crushing is invariably done in a Blake-type (double-toggle) jaw crusher or " sledger." The most popular size in use at present has a feed-opening about 20 x 10 in. With mechanical loading at the quarry face, however, the primary crusher must be capable of crushing the largest lump that can be lifted by the bucket of the loading shovel; the present trend is, therefore, towards the use of larger primary crushers, ranging in size up to 72 x 54 in., although sizes above 54 x 42 in. are uncommon in this country.

6·45 The output from large primary crushers consists of about 6 in. down material, or even 12 in. down in the larger crushers. For secondary and subsequent crushing, jaw, gyratory, cone, impact, disc or roller crushers may be used. A discussion of the characteristics of these different types of crusher is outside the scope of the present book, but for the road engineer it is sufficient to know that all are capable of producing aggregates of the desired " cubical " shape if operated correctly. The main conditions that favour the production of cubical aggregate are: a low reduction ratio at each stage of crushing, removal of the poor-shaped chippings and fines formed in primary crushing, and choke-feeding all secondary and subsequent crushers, in closed circuit with the rejects from the screens. (" Closed circuit " is illustrated for the granulators in Fig. 6·1.) Neglect of these conditions, and particularly the

use of a high reduction ratio to reduce the number of stages of crushing, is the commonest cause of flaky aggregates.

6·46 In a few highly mechanized quarries with large outputs (100,000 tons per year and upwards), as many as four stages of crushing may be employed, all but the primary stage working with a low reduction ratio, choke feed and closed circuit. On the other hand, a high proportion of road aggregates is produced in relatively small quarries with the most primitive plant; this is the principal reason for the flaky, splintery aggregates often supplied, particularly in the smaller (chipping) sizes.

6·47 SCREENING. The crushed rock is sorted into the various aggregate sizes by means of screens. Several different types of screen are in common use—rotary, oscillating and vibratory, horizontal and inclined, square and round apertures. The characteristics of the different types affect quarry economics rather than aggregate quality, for aggregates can be accurately graded on any type of screen that is accurately made, maintained in good condition and not overloaded in use. The commonest cause of badly graded aggregates is overloading the screens.

6·48 All B.S. sizes for aggregates are now based on square-aperture test sieves. Commercial round-holed screens of the type often used in quarries have an effective aperture size of only about 0·8 that of a square hole of the same nominal size; this is another common cause of incorrectly graded aggregates. Nevertheless, the actual size and shape of the commercial screen used to produce an aggregate of given nominal size rests on the judgement and experience of the quarry manager or plant designer. He may find it most economical, for example, to produce ½-in. to ⅜-in. chippings by screening on a ⅝-in. round and a ⅜-in. slotted screen.

6·49 The gradings of aggregates for different roadmaking purposes are given in the British Standards listed at the end of the present chapter. It can safely be said that all these aggregates can be economically produced to specification, and that considerable latitude is possible in the types and sizes of the screens used to produce them. Engineers should appreciate, however, that the crushing process sets a limit to the proportion of any one size that can be economically produced, and that the quarry manager has to find a market for all the sizes. The smaller the aggregate size in popular demand, the higher is the proportion of useless dust produced in meeting the demand. Modern methods of crushing permit the economic production of as much as 40 per cent of a single aggregate size such as ½ to ⅜ in.

6·50 GRADING TO SPECIFICATION. From the screens the sized aggregates are passed to storage bins or stock piles. The range of aggregate sizes produced depends on the markets supplied. The present tendency is to produce closely graded " single " sizes to B.S. 63, " Sizes of roadstone and chippings," and to prepare the graded aggregates required for concrete and bituminous work by mixing the single sizes in appropriate proportions.

6·51 In proportioning the graded aggregates it is common practice to weigh or measure the required quantities from each bin, the judgement of the operator often playing a big part in the process. Some of the larger quarries, however, have plant for carrying out this operation automatically. Briefly, this plant

consists of a series of cam-operated controls, each control opening the outlet of one bin for a predetermined number of seconds in each minute, the bin outlets discharging on to a common conveyor belt. Tests have shown that these electrically operated proportioning plants achieve a much higher order of accuracy than can be achieved by manual operation.

Gravel Pits

6·52 The processes of winning and preparing gravel aggregates differ from those of quarrying crushed rock in several respects, although similar processes of screening and crushing are employed.

6·53 In a typical gravel pit the mixed sand and gravel is dug from the face by mechanical shovels; no blasting is normally required. The shovels load the gravel into trucks, which tip it into a " flume " or washing pit for the removal of much of the clay, silt and organic matter with which most gravels are contaminated. From the flume the gravel, sand and water are pumped up to the screens, a first separation on an $\frac{1}{8}$-in. or $\frac{3}{16}$-in. screen removing the sand, which is passed to classifiers (sometimes followed by " dewaterers ") where the " sharp " and fine grades are separated by rising currents of water. The sharp sand, with a fair proportion of coarse particles retained on a No. 25 B.S. sieve, is preferred for concrete; the fine sand, mostly passing No. 52 B.S. sieve, is preferred for mortar and plaster work.

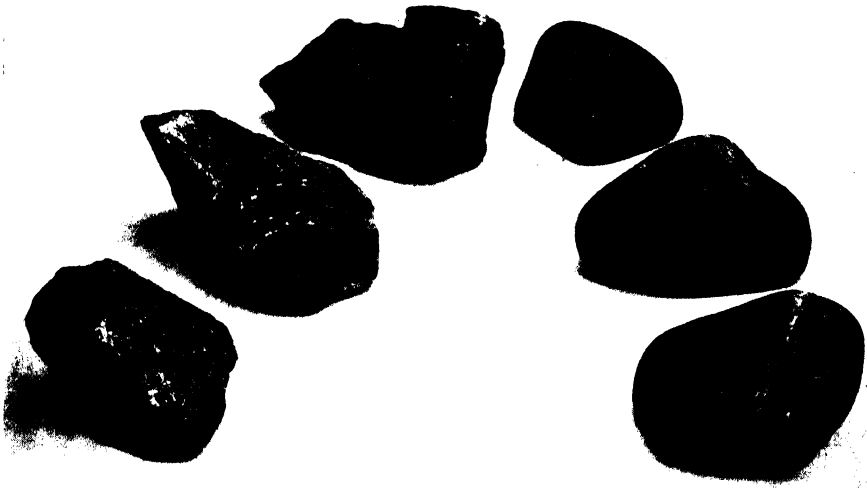
6·54 The coarse aggregate (larger than $\frac{3}{16}$-in.) is sorted into the usual aggregate sizes on screens as in crushed stone quarries, except that as gravel is most often used in concrete, gravel pits usually produce graded aggregates to B.S. 882 and not the single-sized aggregates of B.S. 63. Oversize rejects from the screens, if there are any, are passed through a crusher and returned to the screens.

6·55 Fig. 6·2 is the flow sheet of a typical modern gravel plant, but the flow sheets of other plants may differ considerably from this according to the nature of the gravel worked, the total output and the market supplied. In some pits, for example, the gravel is dredged from beneath water, and in such cases little or no further washing is required. In other pits the removal of clay and silt is a major problem.

Slag

6·56 Most of the slag used in roadmaking is the product of the smelting of iron ore in blast furnaces. The banks of this slag that accumulated during the last century have mostly been used up, and the slag at present used in road construction comes almost entirely from current production.

6·57 There is considerable variety in the methods of production used at different plants. In a typical plant the molten slag is tapped from the furnace at a point above the outlet for the metal, and is run into ladles of capacity varying from 2 to 15 tons—5 tons is a common size. The slag may be cooled in these ladles, or may be tipped on to a bank or into small troughs, pits or canals, the actual method employed for cooling depending to a large extent on the composition of the slag and its liability to disintegration when cooled slowly. After cooling the slag is inspected by an experienced person who is often able to reject at sight material that will be liable to disintegrate. The

(a) British Standard attrition machine

(b) Sample stones before and after test

BRITISH STANDARD ATTRITION TEST FOR AGGREGATES

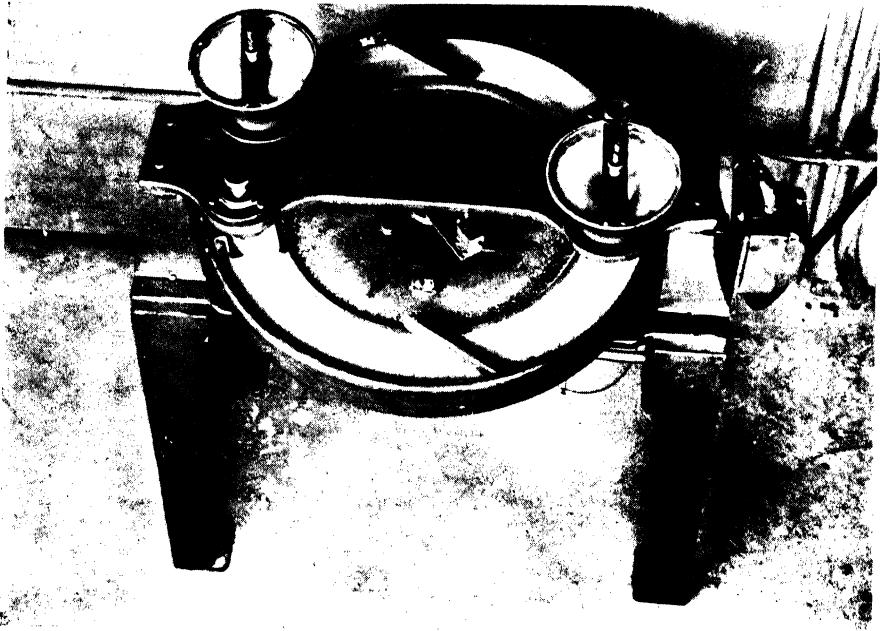
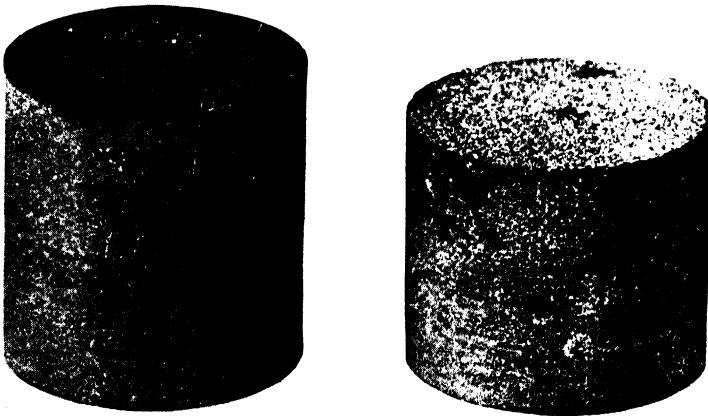
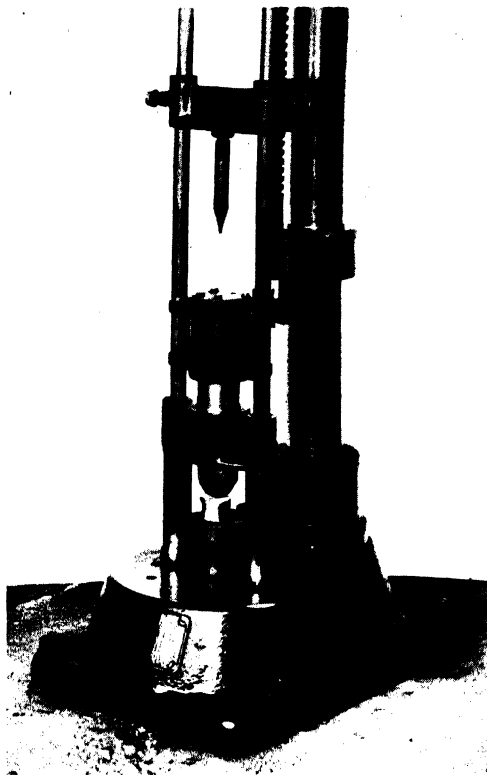
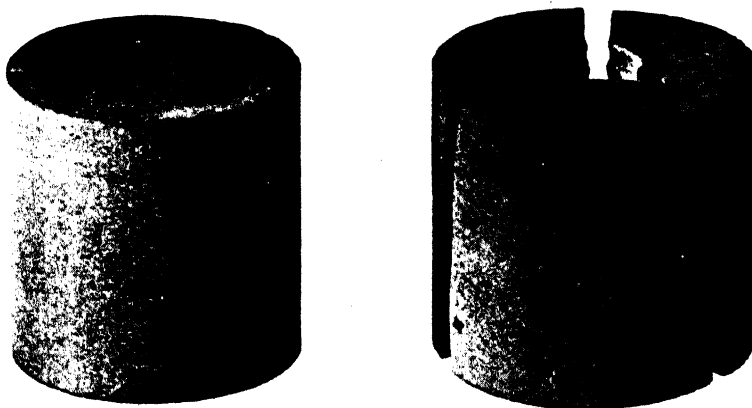



PLATE 6·1

(a) British Standard abrasion machine

(b) Sample before and after test

BRITISH STANDARD ABRASION TEST FOR ROCK SPECIMENS

PLATE 6·2

(a) Impact machine

(b) Sample before and after test

IMPACT TEST ON ROCK SPECIMENS

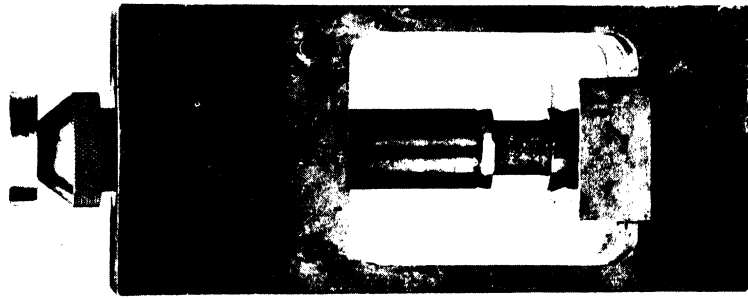
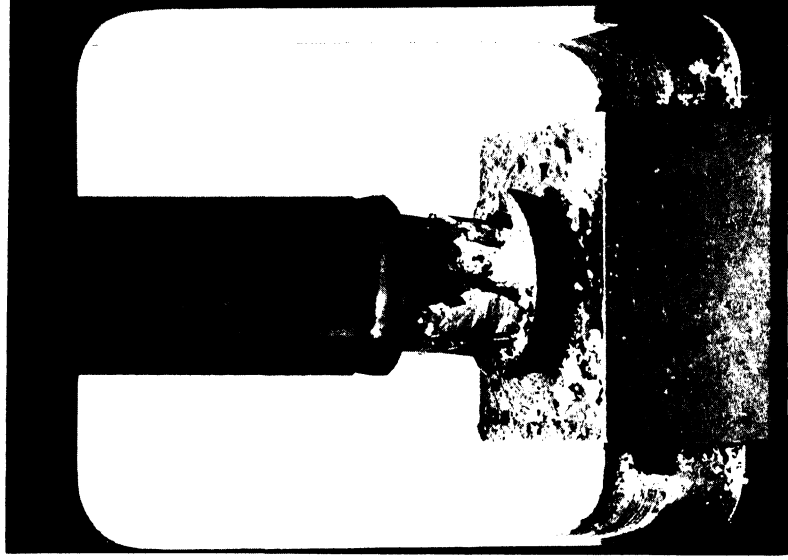



PLATE 6·3

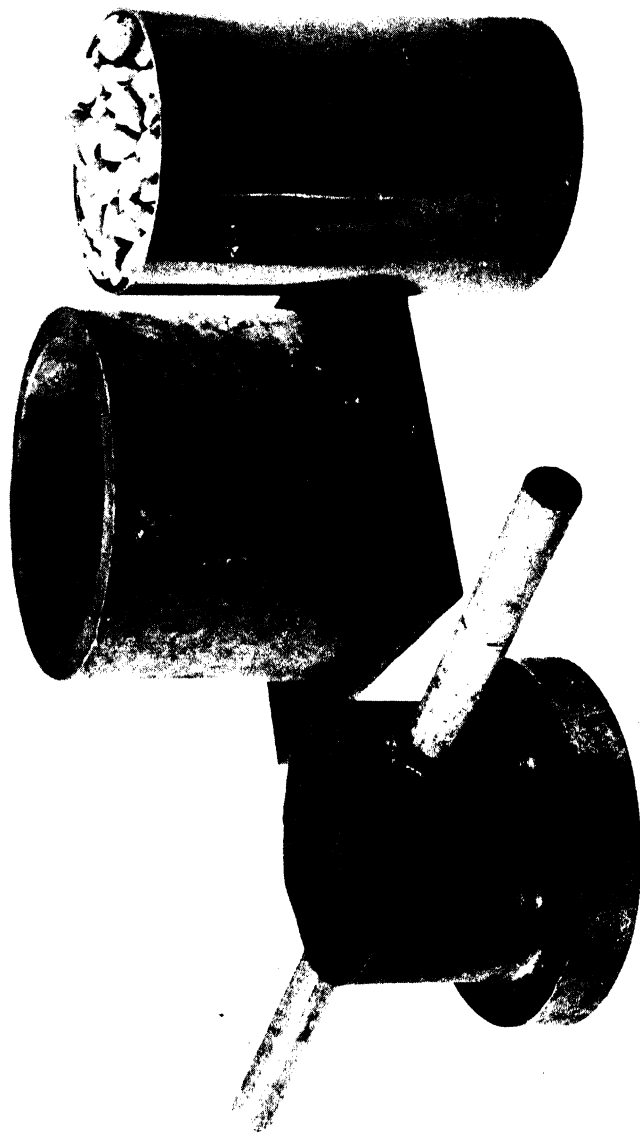
(a) Loading shackles with specimen in position, ready for insertion in compression testing machine

(b) Specimen after test

BRITISH STANDARD CRUSHING STRENGTH TEST FOR ROCK SPECIMENS

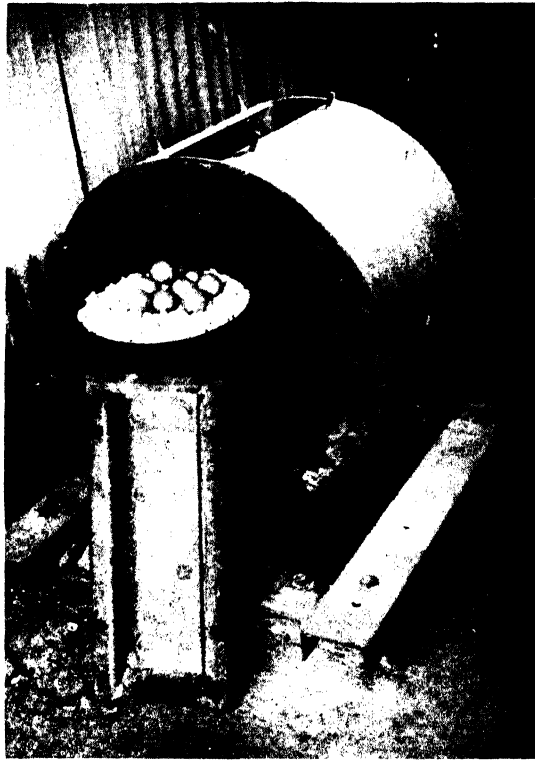
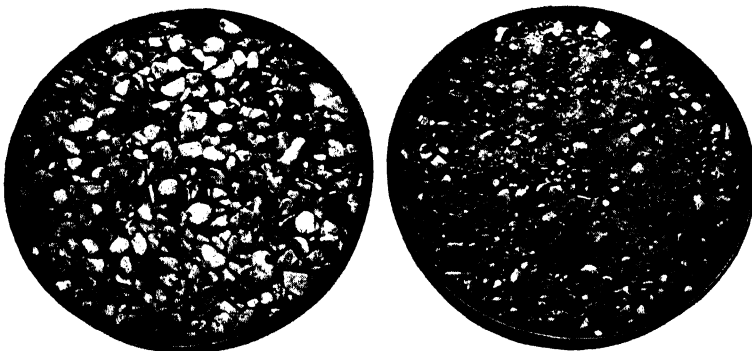
PLATE 6·4

SAMPLE SPLITTER

Riffle box type

PLATE 6·5

BRITISH STANDARD AGGREGATE CRUSHING TEST

Ram-cylinder and test sample

PLATE 6·6

(a) Test machine, test sample and charge of steel balls: cover (in fore-ground) removed from machine to show shelf

(b) Sample before and after test

A.S.T.M. STANDARD LOS ANGELES TEST FOR AGGREGATES

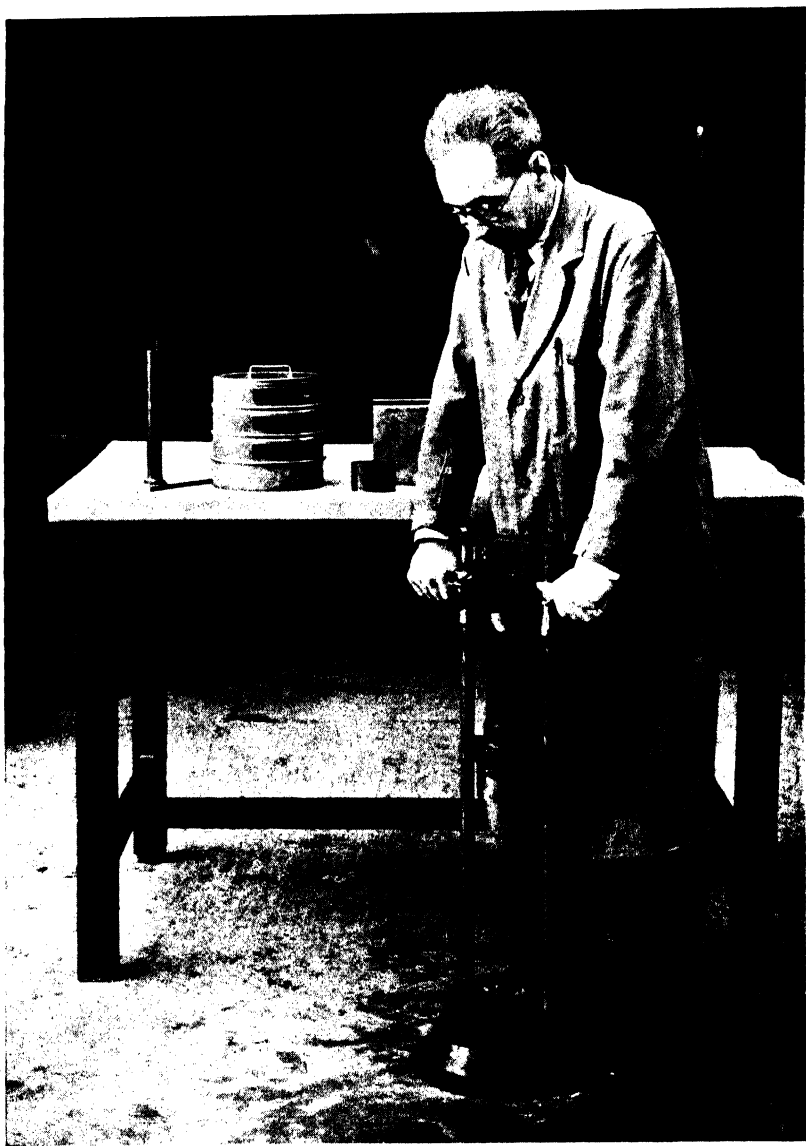
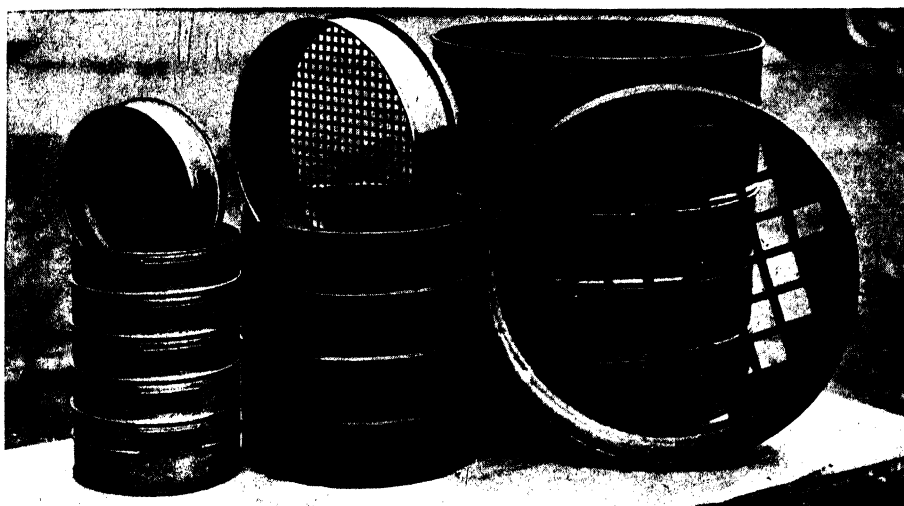
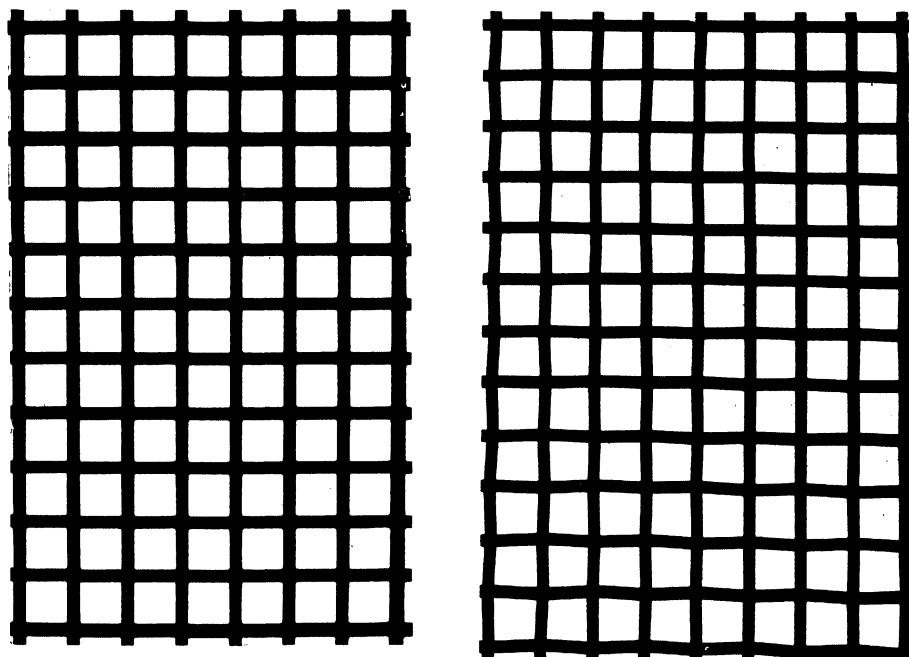
PLATE 6·7

AGGREGATE IMPACT TEST

PLATE 6·8

(A) BRITISH STANDARD TEST SIEVES
Fine, medium and coarse series

(B) COMPARATIVE ACCURACY OF $\frac{3}{8}$-IN. PERFORATED PLATE AND WOVEN WIRE ALTERNATIVES
for the medium-size B.S. test sieves

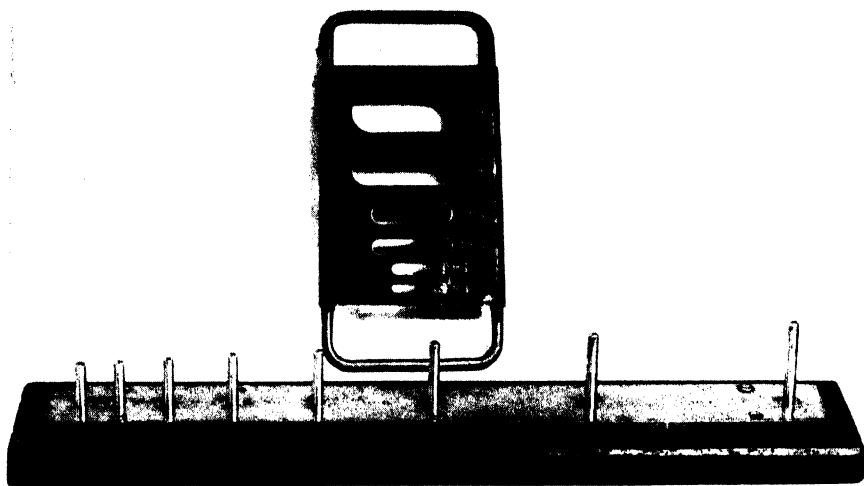



PLATE 6·9

(a) British Standard elongation and flakiness gauges
and flake-sorting sieve

(b) Examples of "cubical", elongated and flaky aggregates

SHAPE TESTS FOR AGGREGATES

PLATE 6·10

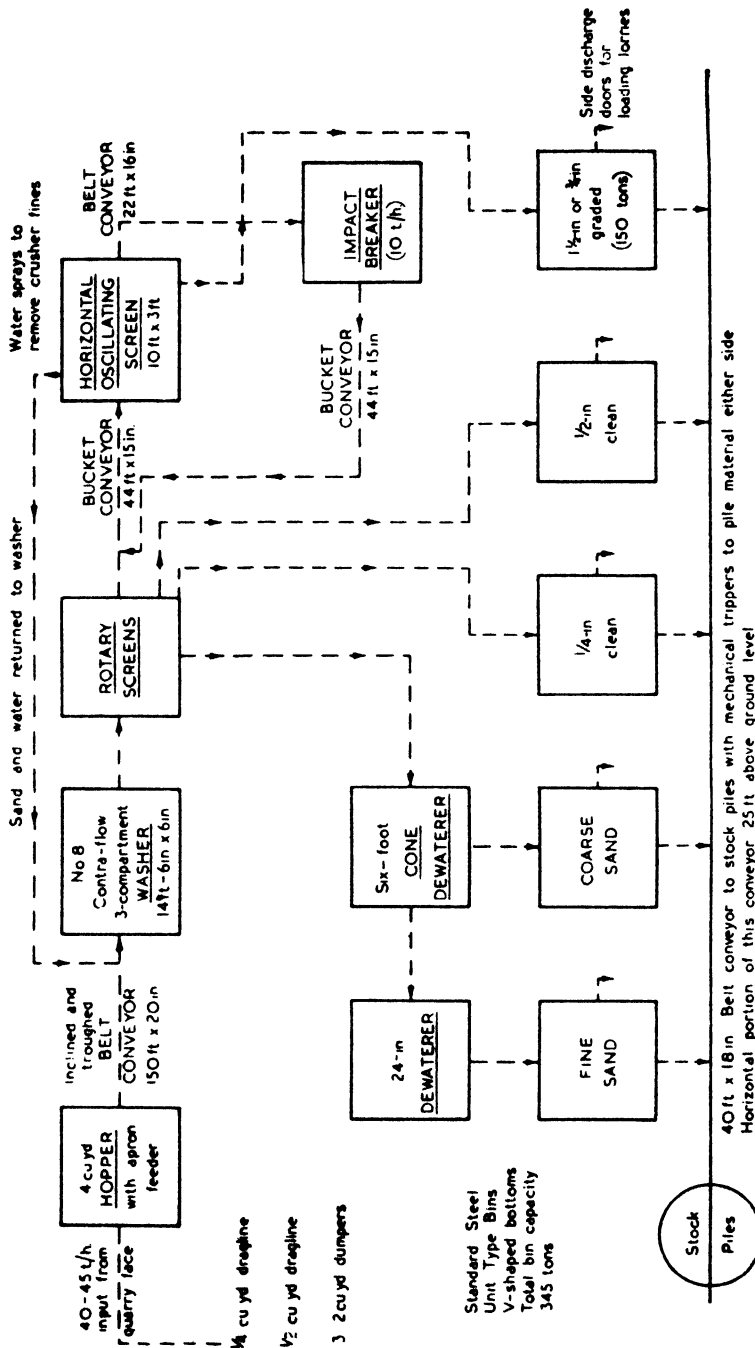



FIG. 6·2 FLOW SHEET OF TYPICAL MODERN GRAVEL PLANT

slag is then "weathered," i.e. exposed to the atmosphere; the periods of weathering used at different plants range from days to years.

6·58 Some preliminary breaking is usually required before the slag is passed to the crushers; this may be done by hand sledging, by a drop ball, by excavators or by blasting. The crushing and screening follow the same lines as in crushed stone quarries. As slag is more widely used in bituminous construction than in concrete, it is most often produced in the single sizes of B.S. 63.

Variability arising in the Production Processes

6·59 The principal causes of variability arising in the production processes are wear on the machinery, the overloading of machinery, and variation in the relative demand for the different sizes of aggregate.

6·60 Wear on crushing surfaces and wear and breakage of the screens is liable to result in an increase in the proportion of oversize particles. Overloading of screens is the commonest cause of an excess proportion of undersize particles.

6·61 Variation in the relative demand for the different nominal sizes of aggregate necessitates alterations in the crushing and screening arrangements. These alterations have considerable effect on the grading and shape of the aggregates produced. It must be emphasized that constant vigilance on the part of the engineer is necessary, if he hopes to maintain reasonable constancy in the grading and shape of the aggregates that he uses.

EVALUATION OF THE QUALITY OF AGGREGATES

6·62 The previous sections have shown that aggregates from different sources differ widely in their composition and properties. Samples of aggregate from the same source may also show considerable variation; knowledge of the rock type of an aggregate, although useful in many respects, is not a sure guide to its properties. It is desirable to carry out tests, not only to compare the properties of aggregates from different sources, but also as a check on the variability of aggregate from a single source.

ROADSTONE TESTS

6·63 Microscopic examination of thin sections of a rock enables the principal minerals to be identified, and the grain size, texture and extent of decomposition of the minerals to be determined. However, the preparation of thin sections, petrological examination and application of the results to the evaluation of roadstone, all call for specialized knowledge that a road engineer cannot reasonably be expected to possess.

6·64 Physical tests are, therefore, desirable to give numerical values for the properties of roadstones, and numerous tests have been developed in many countries. The earlier tests were applied mainly to carefully selected and prepared individual specimens of rock, with the idea of determining the properties of the rock as a roadmaking material. Experience has shown, however,

that the properties of aggregates are often different from those of the massive rock from which they are produced. For this and other reasons dealt with below, recent developments have resulted in tests that can be applied to the crushed aggregates and gravel as used in the road.

6·65 The B.S. tests for roadstone briefly described below include both types of test, and are fully specified in B.S. 812. The American Los Angeles test is also described, because it is widely used in the U.S.A. and elsewhere.

Tests on Rock Specimens

6·66 The attrition test is included in this category because it is one of the older tests, and was originally applied to hand-broken rocks, rather than to crushed aggregates.

6·67 ATTRITION TEST. (See Plate 6·1.) Five kilograms of 2-in. stone are dried at 100 to 110°C. and placed in a closed inclined cylinder which is then rotated at a speed of about 30 r.p.m. for about 5 hours (10,000 revolutions). Wear takes place owing to the movement of the stones over one another and the result of the test is expressed as the percentage of material removed from the sample during test. As this action depends on whether the stones are wet or dry, two different tests are carried out, one with dry stone and one with stone to which water has been added. The results are reported as the dry and wet attrition values respectively.

6·68 ABRASION TEST. (See Plate 6·2.) The machine consists of a steel disc rotating in a horizontal plane. The test specimen is prepared in the laboratory in the form of a cylinder 1 in. in diameter and 1 in. long by drilling and grinding a suitable piece of rock. It is held with its axis vertical and its lower end pressed against the steel disc by a force of 1,250 gm. A supply of 25 to 36 B.S. mesh silica sand is fed continually upon the revolving disc. After the disc has revolved for 1,000 revolutions (about ½ hour) at approximately 28 r.p.m., the loss in weight of the specimen is found and the coefficient of hardness calculated from the formula:—

$$\text{Coefficient of hardness} = 20 - \frac{\text{Loss of weight in gm}}{3}$$

6·69 IMPACT TEST. (See Plate 6·3.) The apparatus consists of a 2-kgm hammer falling freely between vertical guides on to an anvil. Each specimen is prepared as for the abrasion test. It is placed on the anvil with its axis vertical. A small steel plunger, with its lower end hemispherical, rests on the specimen and repeated blows from the hammer are applied to the specimen through this plunger. The height of fall of the hammer is increased progressively in steps of 1 cm. until the specimen fails. The fall, in centimetres, of the hammer for the blow causing failure is taken as the measure of resistance to impact. This test has now been superseded as a British Standard by the Aggregate Impact Test.

6·70 CRUSHING STRENGTH. The test specimens are cylinders prepared as for the abrasion test. The test is carried out by crushing the specimens in a compression testing machine, spherical seatings being used to ensure axial loading of the specimen (see Plate 6·4.) The results given are the cracking stress and the crushing stress in pounds per square inch.

6·71 WATER ABSORPTION AND DENSITY. Tests are made on each of three stones by drying for 72 hours and then immersing in water for 72 hours. The density is calculated from the dry weight of the stone and from the apparent loss of weight of the stone when weighed in water. The water absorption is calculated from the difference in weight between the dry and the saturated stone. This test has been found unsatisfactory in practice, and has been withdrawn from the British Standard.

Tests on Aggregates

6·72 SAMPLING OF AGGREGATES FOR TEST PURPOSES. If the samples of aggregate submitted for test are not truly representative of the bulk, the test results will be worthless. Detailed instructions for sampling aggregates are given in B.S. 812. The essential points of these instructions are: samples should preferably be taken at the time of loading or unloading, taking small samples at regular intervals and mixing them to make a large composite sample, which is finally reduced to the required size by quartering or by means of a sample splitter; if samples must be taken from a bin or stock pile, small samples should be taken from places evenly distributed over the pile, removing about a foot of material from the surface before taking the sample and avoiding any patches of segregated material. The operation known as " quartering " consists in forming a roughly conical heap from the sample, dividing it into four quadrants and taking two diagonally opposite quadrants to form the reduced sample, which is further reduced by the same method until of the required size. Sample splitters are of various types and sizes. A common type—the " riffle box "—is illustrated in Plate 6·5. It consists of a number of narrow chutes discharging alternately on opposite sides of the box; the sample is poured evenly over the top of the box and is thus split into two representative halves, which are collected in boxes placed under the chutes. The operation is repeated until the desired size of sample is obtained. Two sizes of riffle are normally required, one for coarse aggregate and the other for fine.

6·73 AGGREGATE CRUSHING TEST. (See Plate 6·6.) About 7 lb. of $\frac{1}{2}$- to $\frac{3}{8}$-in. aggregate are dried at 100 to 110°C. and placed in a hardened steel cylinder of 6-in. diameter with closely fitting ram or plunger. A load of 40 tons is then applied to the aggregate by means of a compression testing machine. The test result is expressed in terms of the percentage fines (passing No. 7 B.S. sieve) formed. Other nominal sizes of aggregate can be tested if $\frac{1}{2}$- to $\frac{3}{8}$-in. aggregate is not available, but this size is preferred.

6·74 LOS ANGELES ABRASION TEST. (See Plate 6·7.) This test is carried out in accordance with the A.S.T.M. Standard C. 131-47. A 5-kgm sample of aggregate, together with 12 or so steel balls of about $1\frac{7}{8}$-in. diameter, are placed in a steel cylinder 28 in. in diameter fitted with an internal shelf. The cylinder is rotated at a speed of 30 to 33 r.p.m. for 500 revolutions, and the result of the test is expressed as the percentage by weight of the fines (passing No. 12 U.S. sieve, equivalent to No. 10 B.S. sieve) formed. There is a choice of a number of different gradings for the test sample, and the number of steel balls used is adjusted for each grading so as to make the result independent of the grading of the test sample.

6·75 AGGREGATE IMPACT TEST. About $\frac{3}{4}$ lb. of $\frac{1}{2}$- to $\frac{3}{8}$-in. aggregate is subjected to 15 blows from a 30-lb. hammer falling from a height of 15 in. The

result is expressed as the percentage fines passing a No. 7 B.S. sieve. The fines can be measured volumetrically, and the test machine and accessories, which are illustrated in Plate 6·8, are portable.

6·76 AGGREGATE ABRASION TEST. This test is carried out on about 100 gm of ½-in. chippings in the machine used for the British Standard abrasion test described above. The chippings are held by pitch in a shallow tray, and are pressed against the rotating steel disc by a weight of 2 kgm for 500 revolutions at 28 r.p.m., standard 25-36 mesh silica sand being fed continually on to the disc. The result of the test is the percentage loss in weight of the chippings by abrasion; the results obtained with typical roadmaking aggregates vary between 2 per cent for the hardest and 35 per cent for the softest.

6·77 WATER ABSORPTION, SPECIFIC GRAVITY AND DENSITY. Tests are made on 7-lb. samples of crushed aggregate or gravel, by drying at 100 to 110°C. for 24 hours and then immersing in water for 24 hours. The specific gravity is calculated from the dry weight of the aggregate and its apparent loss of weight in water. The density in pounds per cubic foot is calculated by multiplying the specific gravity by 62·4. The water absorption is calculated from the difference in weight between the dry and the saturated stone. In the case of fine aggregate or sand, a sample of the aggregate is weighed (1) as received, (2) after drying for 24 hours at 100 to 110°C., (3) after soaking in water for 24 hours and (4) immersed in water in a pycnometer of known capacity. From these figures the moisture content, water absorption and specific gravity are calculated on either a dry or saturated basis according to the purpose for which the results are required.

6·78 The water absorption of an aggregate is usually accepted as a measure of its porosity and sometimes as a measure of its resistance to frost action, but little work has been done on this subject.

6·79 OTHER PHYSICAL PROPERTIES. There are at present no generally accepted tests for measuring the thermal and elastic properties of aggregates. The effects of heat, frost and other weathering agents, or of production processes such as crushing and drying, on the mechanical strength of aggregates have not been extensively studied, and there is a wide field for research into the significance of these factors.

6·80 SIEVE ANALYSIS. (See Plate 6·9A.) The grading of aggregate is determined by shaking it for not less than two minutes on each of such B.S. square-aperture test sieves (B.S. 410) as are appropriate to define the aggregate size. The results are reported as either the total percentage passing each sieve or the percentages retained between successive sieves; the former method is more convenient for graphical presentation of a grading and is being increasingly employed in specifications for aggregates. Perforated plate sieves are much more accurate than woven wire; they are obligatory in the B.S. coarse mesh series (4-in. to ⅛-in. aperture size) and are given as alternatives to wire mesh in the medium mesh series (½-in. to $\frac{1}{16}$-in.), but owing to their greater accuracy are gradually replacing wire mesh for all sizes down to $\frac{1}{16}$-in. (Plate 6·9B).

6·81 SHAPE TESTS. (See Plate 6·10.) The particle shape of aggregates is determined by the percentage of flaky and elongated particles that they contain. These are defined respectively as particles whose least dimension is less than

0·6 of their mean size and whose greatest dimension is more than 1·8 times their mean size. The aggregate is first sorted on B.S. square-aperture test sieves into a number of closely limited particle-size groups—1-¾ in., ¾-½ in. and so on—and each group is tested for length and thickness on the appropriate gauges (Plate 6·10). The flaky particles can be separated more rapidly on slotted sieves of the appropriate dimensions if desired, but in this case it is necessary to have a slotted sieve for each particle-size group, whereas the one thickness gauge covers them all. There is at present no satisfactory test for measuring the relative roundness or angularity of aggregate, and this property is assessed by visual inspection.

6·82 Bulk density (volume weight or unit weight) and voids. The bulk density of aggregate in pounds per cubic foot is determined from the weight of compacted aggregate contained in a standard measure of $\frac{1}{10}$, ½ or 1 cu.ft capacity. The percentage voids is determined from the bulk density (W) and the specific gravity (G_s) by the formula:

$$\text{Percentage voids} = \frac{(G_s \times 62{\cdot}4) - W}{G_s \times 62{\cdot}4} \times 100$$

or, if the specific gravity is not known, the voids may be determined approximately by measuring the amount of water required to fill them, using the standard $\frac{1}{10}$, ½ or 1 cu. ft. measure.

6·83 Tests for silt, clay and impurities in fine aggregates. The proportion of silt, clay and fine dust in aggregates may be determined by wet sieving on a No. 200 B.S. sieve (protected by a No. 14 British Standard sieve), or by sedimentation methods, of which two are British Standard (B.S. 882). The accurate gravimetric determination is an elaborate laboratory test requiring a special sedimentation apparatus, and taking considerable time. It is possible, however, to make an approximate volumetric estimate by simply shaking up 100 ml. of the fine aggregate with water in a 200-ml. measuring glass and leaving to settle for three hours, after which the silt, clay and fine dust is clearly visible as a layer on top of the coarser material. This test is useful as a routine check only, as the percentage by volume is not easily convertible to percentage by weight, the relationship between the two varying according to the particle size and shape of the silt and clay.

6·84 The relative proportion of organic impurities in sand is determined by observing the discoloration produced by the sample on a standard solution of sodium hydroxide. A graduated standard colour chart is given in B.S. 882, and any sand producing a colour darker than No. 3 on this scale is regarded as suspect, and is further tested by comparing the strength of concrete made with it and with a sand of known good quality.

6·85 Tests for the stability of slag. These consist of:—(1) chemical analysis, (2) a test for iron unsoundness which is carried out by soaking 12 pieces of the slag in water for 14 days, after which they should show no signs of disintegration and (3) a test for falling, dusting or lime unsoundness, consisting in microscopic examination of polished surfaces of the slag that have been etched with magnesium sulphate.

Discussion of the tests

6·86 The variability of all natural rocks and gravels and of slags is reflected in the extreme variability of the results of the tests that are carried out on individual specimens. The effect of this on the number of repeat tests that must be made to ensure a mean result truly representative of the sample, can be clearly seen from Table 6·4, which also shows the superiority of tests on aggregates in this respect.

6·87 The preparation of the cylindrical specimens required for the older tests is a costly business. This, coupled with the poor reproducibility of the results and the fact that these results have not been found to correlate closely with the service behaviour of the aggregates, is causing these tests to give place to the newer tests on aggregates. Until more extensive experience is available with the latter, however, the older tests are likely to remain.

TABLE 6·4

REPRODUCIBILITY OF MECHANICAL TESTS ON ROADSTONE

Test	Coefficient of variation (%)	Number of samples that must be tested to ensure 0.9 probability that the mean will be:—	
		within ± 3% of true mean	within ± 10% of true mean
Dry attrition	5.7	10	1
Wet attrition	5.6	9	1
Abrasion	9.7	28	3
Impact	17.1	90	8
Crushing strength	14.3	60	6
Aggregate crushing	1.8	1	—
Los Angeles	1.6	1	—

In Table 6·4 the coefficient of variation is calculated from the formula

$$\text{Coefficient of variation} = \frac{100\,[\Sigma\,(x - \bar{x})^2]^{\frac{1}{2}}}{\bar{x}\,(N - 1)^{\frac{1}{2}}}$$

where $\Sigma\,(x - \bar{x})^2$ = the sum of the squares of the deviations of individual results (x) from the mean of all the results ($\bar{x}$), and N is the number of results.

6·88 With such a wide variety of tests available, some guidance is desirable as to which should be used in evaluating aggregates for different roadmaking purposes. It is never necessary to carry out all the tests on a sample of roadstone. The older tests still have a field of application in testing lump rock, where no crushed aggregates are available. In evaluating an untried source of rock it is probably best to find out as much as possible about its resistance to abrasion, impact and crushing, in addition to its specific gravity and water absorption, before laying down any considerable plant to work it. It has been shown, however, that there is a broad correlation between the results of many of the tests; for example, aggregates that have a good resistance to crushing in general have also good resistance to impact, abrasion, etc., although it is

not possible to forecast the results of one test from those of another, except within wide limits. It is, therefore, evident that for purposes such as the routine control of aggregate from a single source, or for a rough comparison of the mechanical strength of aggregates from different sources, a single test, such as the aggregate crushing test, will often suffice.

6·89 As there is usually good correlation between the two shape characteristics—flakiness and elongation—it is rarely necessary to carry out both those tests.

TABLE 6·5

RANGE OF TEST VALUES OF BRITISH ROADSTONES

(Based on statistical analysis of tests on about 1,200 samples of roadstone)

Test	Percentage of samples having a test value inferior to the stated value								
	10%	20%	30%	40%	50%	60%	70%	80%	90%
Dry attrition value [1]	5.00	4.30	3.80	3.50	3.20	2.95	2.65	2.35	2.05
Wet attrition value [1]	10.10	8.35	7.15	5.95	4.85	4.05	3.35	2.65	1.95
Abrasion value	15.70	16.50	17.00	17.60	18.15	18.50	18.70	18.95	19.20
Impact value	7.3	9.0	10.6	12.2	13.9	15.7	18.2	21.2	25.5
Crushing strength .. (lb./sq.in.)	15,500	19,500	22,500	25,000	27,500	30,500	34,500	39,000	45,500
Aggregate crushing value (½-in. stone) [1]	29.5	25.4	22.1	19.6	17.0	14.7	12.9	11.4	9.7

[1] For these tests higher test values indicate inferior quality.

6·90 Tests such as sieve analysis, shape, bulk density, tests for impurities and the stàbility tests for slag, the results of which are generally required for comparison with specification requirements, should be carried out as a routine operation. The present trend is towards a considerable extension of this routine testing by both producers and consumers, with consequent improvement in the quality of the aggregates.

Application of Test Results

Note.—Where limits are quoted from British Standards in the present section, they are intended to give a general picture of the properties required; no attempt is made to reproduce the standards in full.

6·91 MECHANICAL PROPERTIES. One of the main uses for roadstone tests is the routine control of aggregate quality, in which case the results are used for purposes of comparison only. When there is marked variation in the results of routine tests on aggregate from a single source, steps can be taken to check the variation at the source.

6·92 In the comparison of aggregates from different sources the matter is not so simple. Rock type alone is not a reliable guide to the properties of an aggregate, as can be seen from Fig. 6·3, which shows the percentage of tested aggregates in the main trade groups whose crushing strengths fell below the given values; a wide range of crushing strength occurs in each trade group, and similar curves have shown that this is true of other mechanical properties.

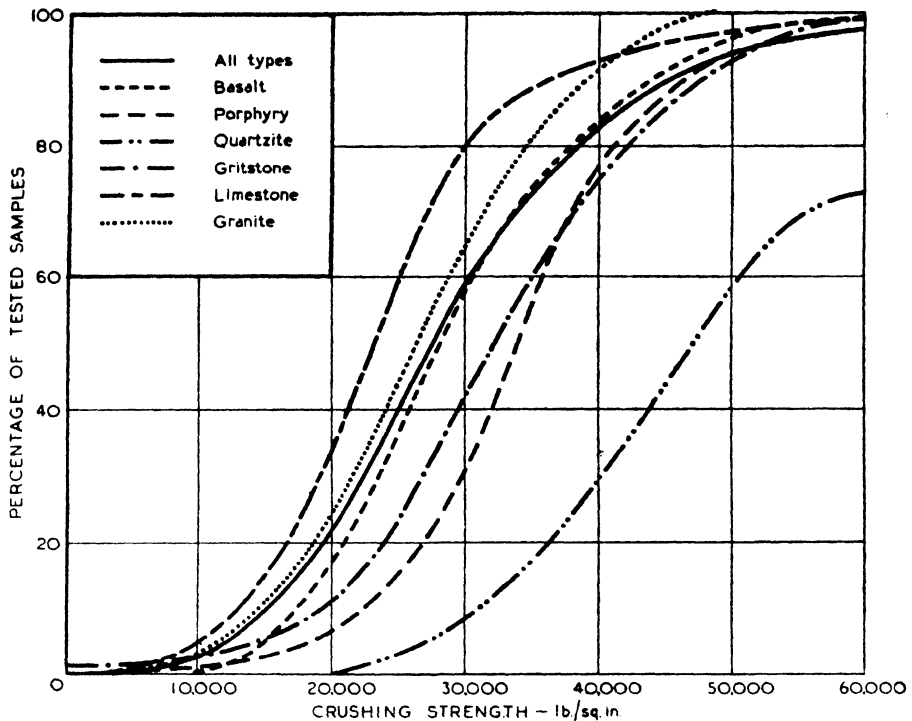



FIG. 6·3 DISTRIBUTION OF CRUSHING STRENGTH VALUES

6·93 By referring, however, to Table 6·5 (which has been constructed from curves similar to those of Fig. 6·3), it is possible to ascertain where a particular test value falls within the range of values obtained from the very large number of representative British roadstones tested at the National Physical Laboratory and the Road Research Laboratory during the past 30 years.

6·94 For example, if a stone had a dry attrition value of 3·50, reference to Table 6·5 would indicate that 40 per cent of the samples tested at the National Physical Laboratory and the Road Research Laboratory were weaker than this particular sample.

6·95 The standard of comparison adopted in Table 6·5 is independent of rock type (granite, limestone, etc.). Thus, a rock that compares unfavourably with others on the basis of this interpretation is not of necessity inferior to normal rock of the same type. By means of Table 6·6, however, the results obtained on any rock may be compared with those obtained on rocks of the same or a similar type. For this purpose the rocks have been classified into Trade Groups (B.S. 812), as in Fig. 6·3. Water absorption and aggregate crushing value have been omitted from Table 6·6 as sufficient data are not available. Values for specific gravity have been added as this gives a useful indication of the quality of a rock when considered in relation to the average specific gravity for the rock type. A low specific gravity, relative to the average for the rock type, indicates partial decomposition in igneous rocks and a lower order of compactness in sedimentary rocks.

TABLE 6·6

RANGE OF TEST VALUES OF STONES IN THE MAIN TRADE GROUPS (B.S. 812)

Test	Percentage of samples having a test value inferior to the stated value								
	10%	20%	30%	40%	50%	60%	70%	80%	90%
				BASALT GROUP					
Dry attrition value ..	4.4	3.8	3.5	3.2	3.0	2.8	2.6	2.3	2.0
Wet attrition value ..	10.8	8.5	6.8	5.4	4.5	4.0	3.5	3.0	2.4
Abrasion value	15.6	16.5	17.0	17.4	17.7	18.0	18.3	18.5	18.7
Impact value	10.0	11.8	13.3	14.8	16.3	18.0	19.8	22.6	27.0
Crushing strength ..	18,000	21,000	23,000	26,000	28,000	31,000	34,000	38,000	44,000
Specific gravity.. ..	2.70	2.76	2.80	2.84	2.87	2.89	2.91	2.94	2.98
				GRANITE GROUP					
Dry attrition value ..	4.0	3.4	3.1	2.9	2.7	2.6	2.4	2.3	2.1
Wet attrition value ..	5.1	4.1	3.5	3.2	3.0	2.8	2.6	2.2	1.8
Abrasion value	18.2	18.6	18.7	18.8	18.9	19.0	19.0	19.0	19.1
Impact value	9.0	10.0	11.0	11.7	12.7	13.8	15.3	17.4	20.3
Crushing strength ..	15,000	19,000	22,000	24,000	26,000	28,000	31,000	35,000	39,000
Specific gravity	2.62	2.63	2.64	2.65	2.66	2.67	2.68	2.68	2.77
				GRITSTONE GROUP					
Dry attrition value ..	4.1	3.7	3.4	3.1	3.0	2.8	2.6	2.3	2.0
Wet attrition value ..	9.0	7.4	6.3	5.5	5.0	4.5	3.9	3.3	2.4
Abrasion value	16.3	17.1	17.6	18.1	18.4	18.7	18.9	19.1	19.3
Impact value	9.0	10.6	12.0	13.3	14.7	16.4	18.6	21.6	25.6
Crushing strength ..	20,000	24,000	27,000	29,000	32,000	35,000	38,000	43,000	48,000
Specific gravity	2.57	2.62	2.64	2.66	2.67	2.68	2.69	2.72	2.75
				LIMESTONE GROUP					
Dry attrition value ..	5.8	5.0	4.7	4.5	4.3	4.1	3.9	3.6	3.4
Wet attrition value ..	11.2	9.8	9.1	8.7	8.2	7.6	6.9	6.0	4.9
Abrasion value	14.4	15.4	15.8	16.2	16.5	16.6	16.8	17.0	17.3
Impact value	5.8	6.8	7.5	7.9	8.4	9.1	10.2	11.6	14.2
Crushing strength ..	13,000	17,000	19,000	21,000	23,000	25,000	27,000	30,000	37,000
Specific gravity	2.62	2.67	2.68	2.68	2.69	2.69	2.70	2.72	2.76
				PORPHYRY GROUP					
Dry attrition value ..	3.8	3.3	3.0	2.7	2.4	2.2	2.0	1.9	1.8
Wet attrition value ..	4.9	3.4	2.7	2.3	2.0	1.8	1.6	1.4	1.2
Abrasion value	18.4	18.7	18.9	19.0	19.1	19.2	19.3	19.3	19.4
Impact value	13.5	15.7	17.7	19.4	21.0	22.6	24.4	27.1	29.6
Crushing strength ..	23,000	27,000	30,000	32,000	34,000	36,000	38,000	41,000	46,000
Specific gravity	2.56	2.59	2.62	2.64	2.65	2.67	2.69	2.72	2.75
				QUARTZITE GROUP					
Dry attrition value ..	3.8	3.1	2.7	2.4	2.2	2.1	1.9	1.9	1.7
Wet attrition value ..	5.5	4.0	3.1	2.7	2.3	2.0	1.8	1.6	1.4
Abrasion value	18.0	18.5	18.7	18.9	19.0	19.1	19.3	19.4	19.7
Impact value	8.6	11.0	13.0	14.8	16.3	18.1	20.1	22.3	25.0
Crushing strength ..	32,000	37,000	41,000	44,000	47,000	51,000	57,000	—	—
Specific gravity	2.54	2.58	2.60	2.61	2.61	2.62	2.63	2.64	2.68

6·96 When some absolute criterion of comparison is required as would be the case if specification limits were to be set to the properties of aggregates, several difficulties arise. Specification limits should be based on a thorough knowledge of the relation of the test results to the service behaviour of the tested aggregates, and published work shows that up to the present very little has been done in this field. Such work as has been done suggests that the correlation of test results and service behaviour is poor for the older tests, especially those carried out on individual specimens, but better for the newer tests that are carried out on aggregates.

6·97 Existing knowledge on this subject is insufficient to form the basis of limits for the acceptance or rejection of aggregates. American work suggests

that the upper limit of the Los Angeles value of aggregates should be 50 for concrete and 40 for bituminous construction. These correspond approximately with aggregate crushing values of 40 and 30 or crushing strengths of 12,000 and 17,000 lb./sq. in. respectively. These figures are in general agreement with British experience and are useful as an approximate guide to the properties required. It should be borne in mind, however, that a higher standard of quality is required in aggregates for the wearing course, especially on heavily trafficked roads, which may require an aggregate with a crushing strength of 30,000 to 40,000 lb./sq. in. (aggregate crushing value between 17 and 12) according to the type of construction. These figures do not, however, apply to slag, the requirements for which are somewhat different from those for other roadmaking aggregates and are dealt with in the following section.

6·98 THE PROPERTIES OF SLAG. Experience suggests that a lower mechanical strength is permissible in slag than in other roadmaking aggregates. Slag with an aggregate crushing value as high as 30 is known to have given excellent service in carpet coats and asphalt. Slag is more likely to fail from chemical instability than from mechanical weakness, and the British Standard requirements for the chemical stability of slag for concrete aggregates (B.S. 1047) are:—

(1) SULPHUR UNSOUNDNESS—Chemical analysis should show that:—
The acid soluble sulphate expressed as $SO_3 < 0{\cdot}7$ per cent.
The total sulphur $< 2{\cdot}0$ per cent.

(2) IRON UNSOUNDNESS—The slag shall be free from signs of disintegration after soaking in water for 14 days.

(3) " FALLING," " DUSTING " OR " LIME " UNSOUNDNESS—If the chemical composition of the slag satisfies either of the following conditions, it is regarded as free from this type of unsoundness:—

$$(\%CaO) + 0{\cdot}8(\%MgO) < 1{\cdot}2(\%SiO_2) + 0{\cdot}4(\%Al_2O_3) + 1{\cdot}75(\%S) \text{ or:—}$$

$$(\%CaO) < 0{\cdot}9(\%SiO_2) + 0{\cdot}6(\%Al_2O_3) + 1{\cdot}75\,(\%S).$$

Slags that fail to satisfy either of these requirements are further tested by microscopic examination of etched polished surfaces, unstable slags being distinguished by characteristic markings.

(4) POROSITY—This is controlled by specifying an upper limit of 10 per cent for water absorption and a lower limit of 78 lb. for the weight of a cubic foot of the compacted aggregate.

6·99 British Standard 802 (Tarmacadam) follows British Standard 1047 generally as to requirements for slag, except that 2·75 per cent total sulphur is permitted and water absorption is limited to 4 per cent.

6·100 THE CLEANLINESS OF GRAVEL AND SAND. The British Standard requirements for the cleanliness of gravel and sand are:—

	B.S. 1241 (Tarmacadam)	B.S. 882 (Concrete)
Clay, silt and fine dust (Max. percentage by weight)	2 per cent (fine aggregate only)	Coarse aggregate 1 per cent. Natural or crushed gravel sand 4 per cent. Crushed stone sand 10 per cent.

Organic impurities	Not sufficient to make the sodium hydroxide solution darker than the standard colour.

Comparison with these requirements is the normal application of cleanliness tests, but they are also being employed to an increasing extent in the more progressive gravel pits, as a routine check on production.

6·101 GRADING AND SHAPE. The grading required in a road aggregate varies according to the purpose for which it is to be used, and the results of sieve analysis are normally required for comparison with the appropriate specification. The particle shape is always required to be as good as possible, but British Standard practice makes allowance for the fact that, with normal crushing and screening, the particle shape deteriorates in the smaller sizes, as can be seen from the following limits from British Standard 63 (Size of roadstone and chippings).

Nominal size of aggregate	*Maximum permitted elongation index* (%)
$2\frac{1}{2}$ in. and 2 in.	35
$1\frac{1}{2}$ in., $1\frac{1}{4}$ in., 1 in. and $\frac{3}{4}$ in.	40
$\frac{1}{2}$ in. and $\frac{3}{8}$ in.	45

B.S. 1241 specifies a *flakiness* index not exceeding 30 per cent irrespective of the aggregate size. (*Note.* In a normal crushed aggregate the elongation index is about 1·5 times the flakiness index.)

6·102 These limits for aggregate shape, like the grading tolerances in B.S. 63, have been based on the results of numerous tests carried out on normal samples of aggregate, and compliance with them should involve no hardship to the aggregate-producing industry.

6·103 In conclusion it must be admitted that the overriding factor in the selection of aggregates is often the economic one, principally the cost of transport. It is sometimes necessary to use a local aggregate of known indifferent quality because the cost of haulage of a better-quality—but more distant—aggregate would be prohibitive. The minimum standards of quality permitted are, however, quite low, and it is doubtful if anywhere in this country need be out of reach of aggregates complying with British Standards.

SUMMARY

6·104 The modes of formation of the natural rock, gravel and slag from which road aggregates are prepared are briefly described, together with the production processes. It is shown that the modes of formation may cause variability in the composition, texture and physical properties of aggregates, and that the production processes may cause variability in their grading, shape and cleanliness. The tests that can be applied to control this variability and to provide comparisons between aggregates from different sources are briefly described and discussed. The present trend is towards the development of *ad hoc* tests that can be applied to aggregates as used in the road; such tests have better reproducibility and correlate better with the service behaviour of the aggregates than tests on prepared specimens. Existing knowledge of this correlation is not sufficient to justify the application of rigid specification limits to the physical properties of aggregates, but approximate limits are

given as a general guide. By the use of tables based on statistical analysis of large numbers of test results, it is possible to compare aggregates with each other and with the general run of roadstones. The requirements for slag, which are somewhat different from those for other roadmaking aggregates, are also described.

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B.S. 882 Concrete aggregates and building sands from natural sources.

B.S. 1047 Blastfurnace slag coarse aggregate.

B.S. 63 Sizes of roadstone and chippings.

B.S. 802 Tarmacadam and tar carpets: granite, limestone or slag aggregate.

B.S. 1241 Tarmacadam and tar carpets: gravel aggregate.

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CHAPTER 7

CHALK EMBANKMENTS AND SUBGRADES

INTRODUCTION

7·1 The behaviour of chalk when used as a foundation material is of particular importance to road engineers in southern England.

7·2 In the absence of freezing conditions, chalk provides a very stable subgrade over which only a few inches of construction are necessary for main road traffic. Unfortunately most varieties are susceptible to frost heave, and the thickness of construction will normally be determined by the maximum probable depth of frost penetration, and not by the bearing capacity of the chalk. On secondary roads, where it may cost too much to lay a thickness of surface material sufficient to exclude frost from the chalk, measures to minimize the damage to the surfacing must be taken during the thaw.

7·3 Whilst soil tests such as the liquid and plastic limit tests can be applied to powdered chalk the results have little significance since the material does not normally occur in this condition. The problems associated with the compaction of broken chalk—a matter of importance in connexion with the construction of chalk embankments—are however very similar to those which arise in the case of other soils and experimental work shows that the technique of compaction developed for soils can be applied to chalk fill.

CHARACTERISTICS OF CHALK

7·4 Chalk is a soft limestone consisting principally of the remains of marine organisms. It has a rigid porous structure, and a hardness which varies considerably, depending on the type. The hardness can be arbitrarily related to the porosity, which in turn can be expressed in terms of the saturation moisture content. For a very hard chalk the saturation moisture content is between 5 and 10 per cent, whilst values of 25 to 30 per cent are found in the case of the softest varieties.

7·5 The upper strata of virgin chalk are normally fissured with fine cracks, which divide the chalk into lumps which vary in size from a few inches across at the surface to several feet across at a depth of about 10 ft.

7·6 If dry chalk is wetted, surface tension forces cause the water to be drawn into the structure. The moisture content of chalk in its natural condition is always close to the saturation value, except in the surface layers subject to evaporation. It should be noted that the term " moisture content " has a rather different significance in the case of a mass of broken chalk from that which it has when applied to a soil made up of relatively impermeable particles. In crushed chalk the water is contained inside the particles rather than on their surface.

7·7 Chalk does not swell or shrink appreciably with changing moisture content.

STRENGTH OF CHALK AS A SUBGRADE MATERIAL

7·8 In the absence of frost, chalk provides a very stable subgrade material. This applies to chalk in the undisturbed condition and also to recompacted chalk, provided adequate attention is paid to the compaction process. Experience has shown that for roads not affected by frost, a riding surface of only a few inches in thickness is all that is necessary for main road traffic. For this reason no serious attempt has so far been made to apply normal methods of pavement design to chalk subgrades.

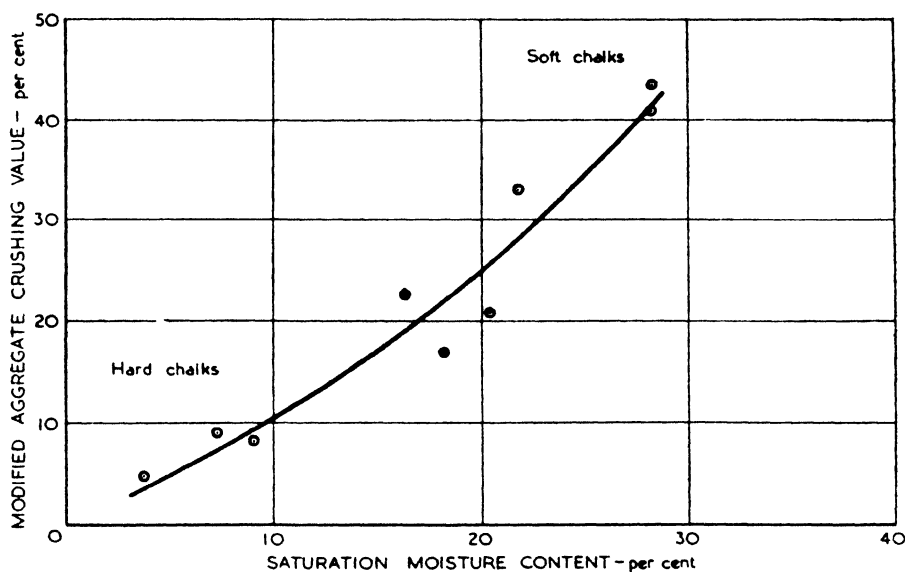



FIG. 7·1 MODIFIED AGGREGATE CRUSHING VALUE RELATED TO THE SATURATION MOISTURE CONTENT FOR DIFFERENT VARIETIES OF CHALK

7·9 The exact manner in which a chalk subgrade might fail under extremely heavy traffic is a matter of conjecture. Whilst in the case of virgin chalk a reduction in the voids ratio due to crushing would have to occur, relative movement of the chalk lumps might be a contributory cause in chalk fill. A rough guide to the relative crushing strengths of different types of chalk can be obtained from the aggregate crushing test (see Chapter 6), if the load applied is reduced to a quarter of that specified in the standard test. Fig. 7·1, based on such modified aggregate crushing tests carried out on a number of chalk samples, relates the modified aggregate crushing value to the saturation moisture content for a number of samples.

COMPACTION OF CHALK

7·10 When broken chalk is subjected to the B.S. compaction test, a dry density/moisture content relationship is obtained similar in shape to that found for ordinary soils. Since chalk crushes under the blows of the rammer

a separate sample must be used at each moisture content studied. The optimum moisture content is of the same order as the saturation moisture content of the individual chalk fragments. Fig. 7·2 shows the relationship obtained for soft chalk samples having different initial gradings.

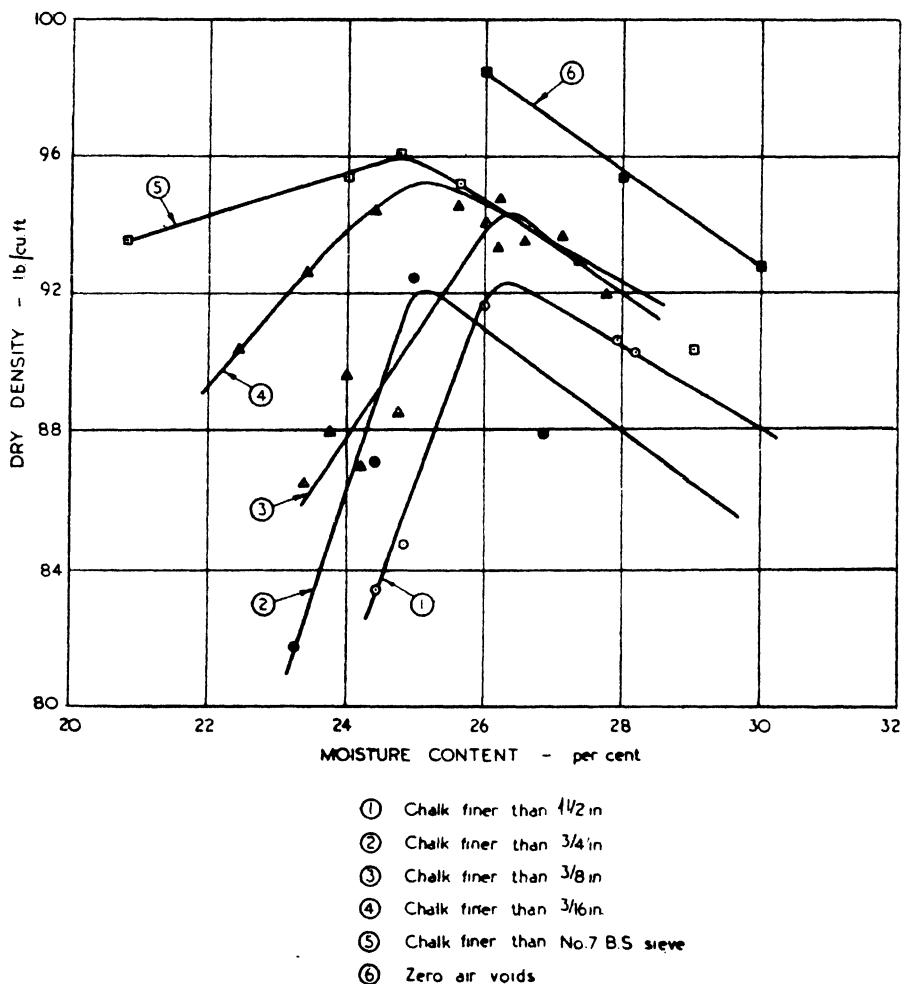
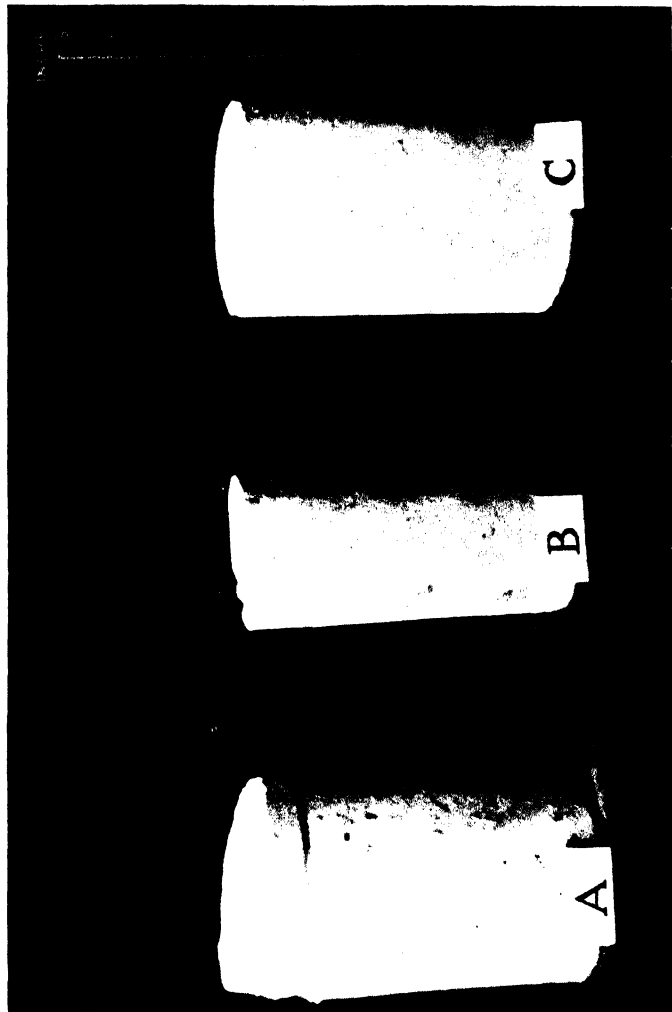



FIG. 7·2 DRY DENSITY/MOISTURE CONTENT RELATIONSHIPS FOR CHALK

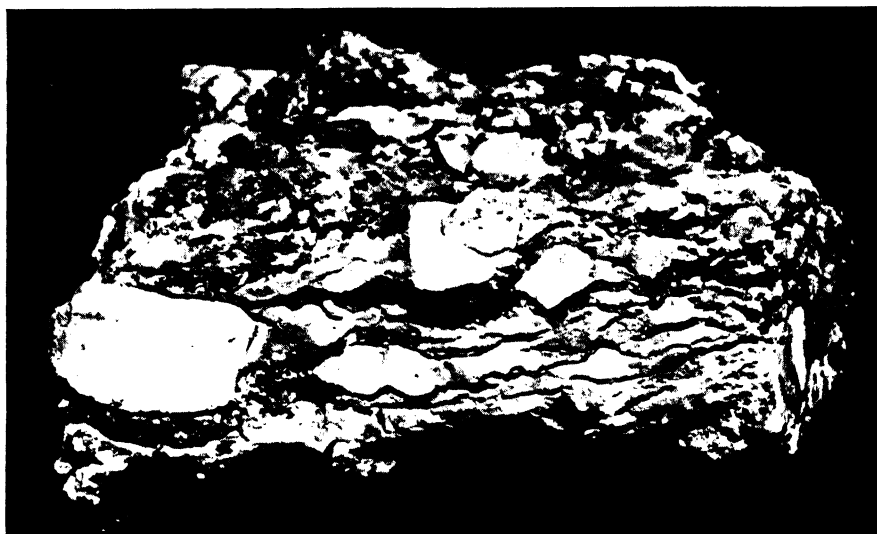
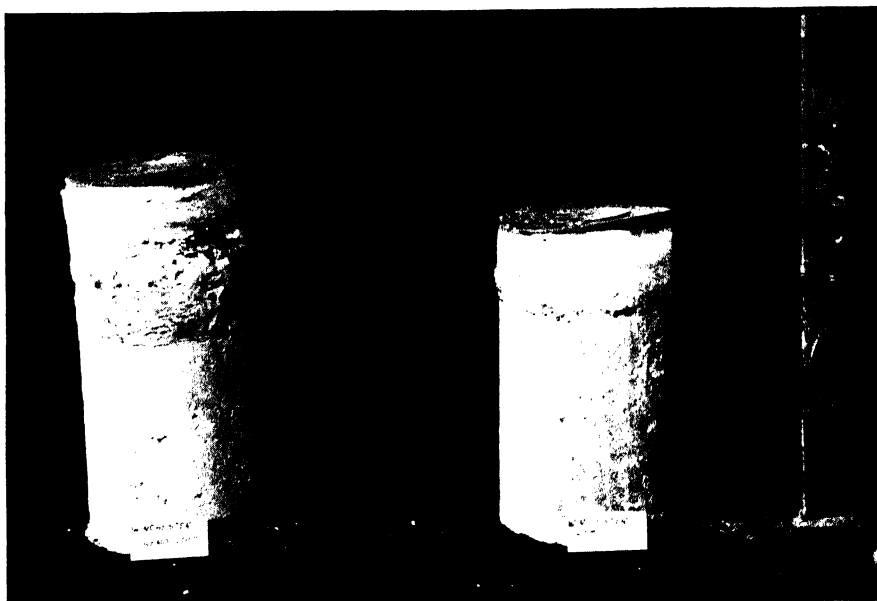
B.S. compaction test

7·11 The construction of large chalk embankments should, where possible, be preceded by full-scale compaction trials with the available equipment. The object of these tests would be to determine the degree of pulverization necessary in the broken chalk, the maximum permissible thickness of each compacted layer and the number of passes of the roller necessary to give a bulk density approaching that of the undisturbed chalk.

A. Soft chalk, 25 per cent moisture content (Winchester, undisturbed soft sample).

B. Hard chalk, 10 per cent moisture content (Steyning, undisturbed sample).

C. Medium chalk, 18 per cent moisture content (Winchester, undisturbed medium soft sample).

SOLID CHALK CYLINDERS AFTER BEING SUBJECTED TO LABORATORY FREEZING TESTS

PLATE 7·1

(A) ICE LENSES IN FROZEN CHALK
removed from a road subgrade

Soft chalk, saturation moisture content 25 per cent.

Medium chalk, saturation moisture content 18 per cent.

(B) FROST HEAVE IN 6-IN. REMOULDED CHALK CYLINDERS SUBJECTED TO LABORATORY FREEZING TESTS

PLATE 7·2

(A) DIFFERENTIAL MOVEMENT DURING FROST BETWEEN SLABS AND CONSTRUCTION STRIP
Concrete road on chalk

(B) DETERIORATION OF ROAD SURFACE SUBSEQUENT TO THAW
Bituminous surfacing on chalk fill

PLATE 7·3

7·12 So far insufficient full-scale work has been carried out to allow recommendations to be made relating to the compaction of all varieties of chalk. During the construction of an 11-ft embankment with soft chalk (saturation moisture content 25 per cent) it was found that dry densities exceeding 90 per cent of the density of the chalk lumps could be obtained with a 10-ton smooth-wheel roller, if the chalk was compacted in 6- to 9-in. layers. The relationship between the number of passes of the roller and the dry densities obtained using 6- and 9-in. layers is shown in Fig. 7·3. It will be noticed that the density

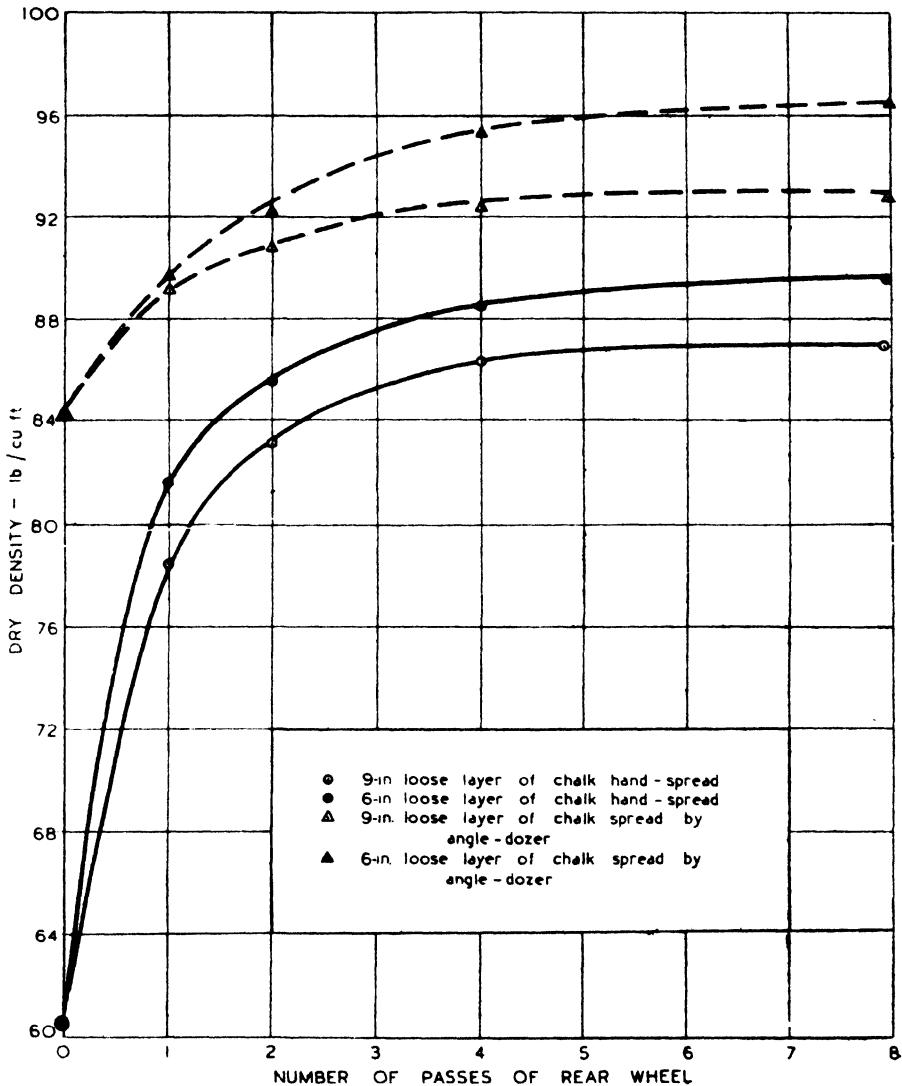



FIG. 7·3 RELATIONSHIPS BETWEEN THE DRY DENSITY OF CHALK AND NUMBER OF PASSES OF A 10-TON SMOOTH-WHEEL ROLLER

Moisture content of chalk 25 per cent

obtained was materially increased when the initial spreading of the chalk was carried out by a bulldozer. This was attributed to the crushing and vibratory action of the tracks. In these tests the chalk was crushed to a maximum size of 6 in. before spreading.

7·13 The question of the settlement of chalk embankments is dealt with in Chapter 24.

FROST HEAVE IN CHALK FOUNDATIONS

7·14 The factors which give rise to frost heave in road foundations will be discussed in Chapter 18. In very soft chalk ice lenses may form in the solid chalk causing cracking across roughly horizontal planes. Plate 7·1 shows cylinders cut from solid specimens of soft, hard and medium chalk, after they had been subjected to a freezing test for several days. In this test the samples were frozen from the surface while their lower ends were in contact with water maintained at a temperature slightly above freezing point. The harder chalks were undamaged but cracking due to the formation of ice lenses occurred in the softest specimen. It is more common for the lenses to form in the fissures between the chalk lumps. This is illustrated in Plate 7·2A which is a photograph of a sample of soft chalk removed from a frozen subgrade. All varieties of fissured or re-compacted chalk suffer from this type of heave, but owing to their greater permeability soft chalks are more susceptible than the harder kinds. The growth of 6-in. cylinders subjected to a freezing test similar to that described above is shown in Plate 7·2B. These samples were re-compacted to a bulk density approaching 90 per cent of the bulk density of the chalk lumps, and were subsequently frozen for 14 days. The heave on a harder specimen having a saturation moisture content of 10 per cent (not shown in Plate 7·2B) was found to be rather less than on the medium chalk.

7·15 The depth of frost penetration under road surfaces is unlikely to exceed 15 in. in the chalk country of southern England. For main roads on soft chalk a thickness of non-frost-susceptible material of about this figure is required if complete freedom from frost heave is required. On harder chalks 10-12 in. should be sufficient. Where roads are affected by frost heave, damage to the surfacing can be minimized if traffic is restricted during the thaw period.

7·16 Examples of frost damage to road foundations, which occurred during the severe winter of 1946-7, are shown in Plate 7·3A and 7·3B. The concrete slabs in Plate 7·3A heaved approximately 2 in. above the central construction strip which was founded on an additional thickness of concrete. In the case of the heavily trafficked road shown in Plate 7·3B, chalk fill had been used 12 in. below the level of the road surface. A complete break-up of the surface occurred during the thaw.

STABILITY OF CHALK SLOPES

7·17 Chalk in the natural state has such a high strength that cuttings can be made with relatively steep slopes (45° to 80°). Frost, however, is likely to cause instability in the surface layers of slopes, and vegetation has been employed to minimize this effect. The vegetation acts as a blanket, which reduces frost penetration into the chalk, Grass can be used on slopes up to 45°, but

on steeper gradients it is difficult to establish a sufficiently thick growth. Ivy, planted in pockets of soil, has been used with success on slopes as steep as 65°.

7·18 Chalk fill used in the construction of embankments behaves in a similar manner to a granular material in so far as the maximum slope angle is equal to the angle of repose. For soft chalk the slope will be about 35° rising to 45° for the harder varieties. Grass can easily be established on such slopes.

PERMEABILITY OF CHALK

7·19 Laboratory experiments show that the saturated permeability of solid chalk is low, being of the same order as that of fine silts. Figures for soft and hard chalks of $2{\cdot}6 \times 10^{-6}$ cm./sec., and 0.5×10^{-6} cm./sec., have been obtained. In practice, however, the fissures will carry away large quantities of water, and the material can be regarded as well drained. On chalk tracks the fissures become clogged with powdered chalk and slurrying of the surface is common in winter.

SUMMARY

7·20 In this chapter the principal properties of chalk likely to be of interest to road engineers are discussed.

7·21 Although chalk subgrades are normally stable enough to carry heavy traffic loads with only a few inches of surface construction, most varieties are liable to frost heave. On main roads, on which even temporary surface movements cannot be tolerated, the thickness of construction must in general be determined by the probable maximum depth of frost penetration.

7·22 Settlement of chalk embankments can be largely overcome if adequate attention is paid to compaction of the fill. Compaction in layers 6-9 in. thick is advocated. Information is given regarding suitable slopes for cuttings and embankments.

CHAPTER 8

SOIL SURVEY PROCEDURE

INTRODUCTION

Object and Scope

8·1 A soil survey forms an essential part of the preliminary engineering survey for a road and its purpose is to furnish the engineer with information regarding soil and ground-water conditions on which a rational and economic design can be based. In general, a soil survey involves:—

(1) The exploration of soil conditions over the site by boring or otherwise, and the preparation of sections* indicating the nature of the ground.

(2) The examination and testing of samples of the soil taken from the site and the reduction of the information to engineering recommendations.

8·2 Table 8·1 outlines the significant information that should be obtainable from a soil survey to enable the following design requirements to be established:—

(1) Suitability of the proposed location, both horizontally and vertically.

(2) Selection of suitable materials for embankments.

(3) Safe gradients for sides of cuttings and embankments.

(4) Earthwork quantities; bulking or shrinkage; volume of rock excavation.

(5) Subsoil and surface drainage requirements.

(6) Need for treatment of subgrade and type of treatment required.

(7) Thickness of carriageway pavement.

(8) Suitability of local materials for use in the construction of stabilized bases.

*As is shown by the following quotation taken from " A Treatise on Roads "(†) published in 1833 and said to describe the practice followed by Telford, the need for soil surveys has been recognized ever since serious consideration has been given to road construction:—

" A vertical section should be made, and the nature of the soil or different strata should be shown over which each apparently favourable line passes, to be ascertained by boring; for it is by this means alone that the slopes at which cuttings and embankments will stand can be determined and calculated . . . . . "

" If bogs or morasses are to be passed over, the depth of the peat should be ascertained by boring and the general inclination of the country for drainage should be marked."

†PARNELL, Sir H. A treatise on roads. London, 1833 (Longman, Rees, etc.), p. 40.

TABLE 8·1

INFORMATION OBTAINABLE FROM SOIL SURVEY AND ADDITIONAL INVESTIGATIONS

Ref. No.	Main design requirements	Particular requirements	Information obtainable from soil survey	Information obtainable from additional investigations
1	Suitability of selected location	Suitability of horizontal alignment	Avoidance of unsuitable ground such as peat, soft clay, areas subject to landslips or rockfalls or to water-logging in winter.	
		Suitability of vertical alignment	Avoidance of high embankments on weak foundations or unstable strata. Reduction or avoidance of cuttings in rock or unstable strata. Maintenance of formation level at a suitable height above water-table level.	
		Stability of foundations under embankments	Need for further investigation indicated. Possibilities of strengthening the bearing capacity of the subsoil.	Shear strength determinations necessary to check designs.
			Indications of probable settlement of pipes and culverts.	Consolidation tests necessary for closer estimates of probable settlement.
		Stability of rock strata	Visual inspection and preliminary geological information give indications of possible landslips and rock slides.	
2	Selection of materials for embankment construction	Suitability of excavated material for embankment construction Selection of borrow pit sites	Soil type gives:— (i) Quality of material as filling, probable optimum moisture contents. (ii) Indication of compacted densities. (iii) Suitability for winter construction. (iv) Desirable types of earthwork equipment.	Compaction tests necessary to give definite information. In conjunction with density tests information on probable bulking or shrinkage of material.
3	Earth slopes	Cross-section of cuttings and embankments	Indications of safe slopes in cut and fill.	In cohesive soils, shear strength of soil required to check designs in doubtful cases.
4	Earthwork quantities	Volume of excavation	Volumes of peat or rock excavation. Allowances for bulking or contraction (these depend on compacted densities, see item (ii) of Ref. 2 above).	Compaction tests desirable for more detailed estimates.
5	Drainage	Subsoil drainage	Ground-water studies indicate need for subsoil drainage and location of drains and interceptors. Spacing of drains depends on soil type.	
		Surface drainage	Drainage or diversion of ponds, streams and springs. Location of catch-water drains.	
6	Preparation of subgrade	Need for subgrade treatment and work required	Depends on nature of soil and season in which construction takes place.	
7	Design of pavement	Type and thickness of base	Indications of thickness required given by soil type and wheel loading proposed.	Special investigations needed for closer estimates of required thicknesses.
8	Stabilization	Suitability of local materials for construction of flexible bases	Indications given of practicability of various forms of stabilized construction.	Special *ad hoc* investigations needed for detailed studies.

8·3 The scope of the recommendations that can be made will depend on the facilities available, which may permit:—

(1) Visual and manual examination alone.

(2) Standard classification tests in addition to visual and manual examination.

(3) Detailed investigation of specific soil properties in addition to classification tests.

Only the first two can properly be regarded as an integral part of the soil survey.

8·4 Detailed investigations of specific properties vary from soil compaction tests to investigations requiring " undisturbed " samples which are matters for specialists. The conditions revealed by a soil survey indicate, as outlined in Table 8·1, where further investigations of this type may be profitable.

8·5 The only site explorations considered in this paper are those of relatively shallow depth that can be carried out with simple hand-boring equipment without the use of lining tubes. Hand-boring equipment is suitable for most borings required for roads, although in some cases lined borings may have to be used. To assist engineers who do not possess soil-testing equipment, reference is made to the information obtainable by manual and visual inspection of the samples of soil. So much additional information can be obtained from soil tests, however, that simple laboratories equipped to carry out classification and compaction tests should be available in connexion with all projects of any magnitude. Such laboratories may be set up in a single room or, as mentioned later, a mobile laboratory may be used.

8·6 It is difficult to give accurate figures of the costs of soil surveys since conditions vary widely but the direct cost (exclusive of travelling) would probably be of the order of £25-50 per mile at the present time where no unusual difficulties are encountered. The cost is therefore very small in comparison with the costs of construction.

Outline of Soil Survey

8·7 A typical soil survey usually provides a vertical section or sections, showing the nature of the soil in different strata, and it should be made in conjunction with the topographical survey. Such a soil survey can be rapidly carried out by the hand-boring methods described below, which for the same expenditure of time and energy enable a much more extensive examination of soil conditions to be made than by the digging of trial pits, which becomes costly and laborious if the pits are closely spaced.

8·8 Fig. 8·1 illustrates a typical result of a soil survey and shows the soil profile on a site with a concealed bed of peat covered by 2 ft to 6 ft of soil. By hand boring it was not possible with the tools available to penetrate the peat, the maximum depth of bore being restricted to about 10 ft owing to repeated collapse of the sides of the holes. Subsidiary borings made on either side of the proposed centre-line showed that it was not practicable to avoid the peat by any reasonable deviation. Lined borings were accordingly put down by a boring contractor to ascertain the total thickness of the peat that it would be necessary to remove or displace. It will be noted that to obtain

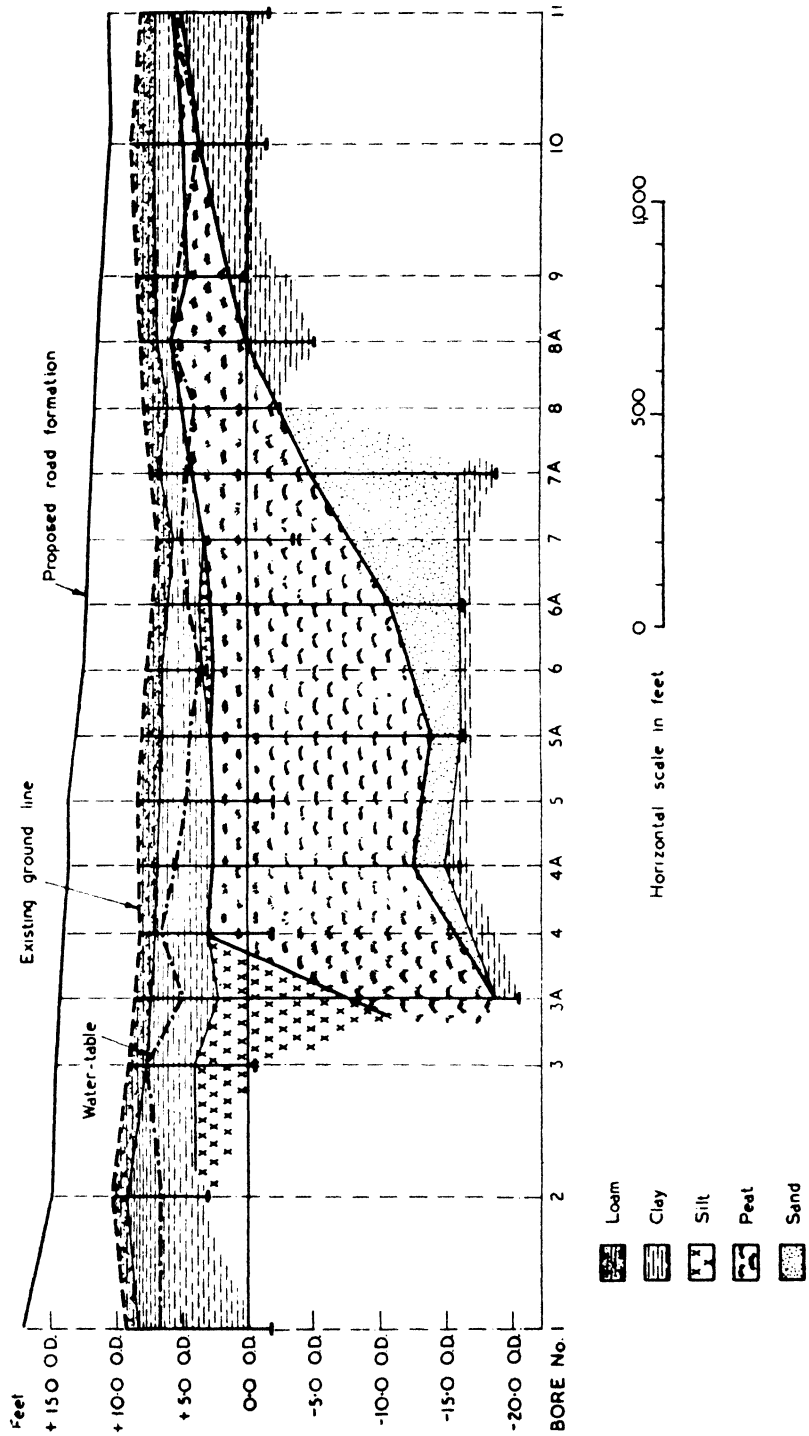



FIG. 8·1 TYPICAL SOIL PROFILE SHOWING CONCEALED BED OF PEAT

additional information the lined borings, marked " A," were interspersed between the original borings. This example illustrates the insurance value of soil surveys against unforeseen foundation conditions.

PERSONNEL AND EQUIPMENT

Personnel

8·9 The field work necessary on a soil survey includes taking borings and obtaining typical soil samples. Between 20 and 400 linear feet of boring per mile are required on the survey of a trunk road, with 200 linear feet as a general figure. To carry out this work over a length of two or three miles of road, one engineer and two or three labourers are required. When working far from the base, the labourers may be hired locally, but when working near the base or on a small job it may be more economical to bring experienced men from the base.

Equipment

8·10 On most soil surveys a light van is essential to carry the necessary equipment to the site. A 10-cwt. van will usually be sufficient for this purpose, since if the weight of samples becomes too large some can be forwarded to the laboratory separately. An alternative procedure is to employ a mobile laboratory, consisting of a 3- to 5-ton lorry or trailer, suitably equipped. Such a laboratory, in addition to being used for the soil survey proper, can also readily be equipped for soil classification and compaction tests so that, if required, the whole work can be carried out on the site.

8·11 Plate 8·1 illustrates the type of mobile laboratory used by the Road Research Laboratory for a complete soil investigation; it includes equipment for all the laboratory and field tests usually carried out except those requiring heavy equipment. This type of laboratory is perhaps too elaborate and costly for many local authorities and Plate 8·2 illustrates a simpler type more suitable for such authorities.

8·12 If a light van is used, it can conveniently be fitted with racks on the walls to carry surveying instruments and ranging rods, and with removable boxes for sample tins, pegs and other small stores. Boxes about 15-in. cube are suitable for this purpose. The sample tins for the soil should be airtight so that no loss of moisture occurs from any of the samples that may subsequently be required for laboratory tests. Except for gravelly soils, a 1½-lb. sample is sufficient for identification tests and this can be conveniently placed in a 1-lb. lever-lid tin 2½ in. high by 3½ in. in diameter. The larger samples required for more extensive tests need not be stored in an airtight tin, but a small part of these samples should be kept airtight for moisture content tests. Typical weights are given in Table 8·2.

8·13 On major road construction if samples are taken at vertical intervals of 1 ft 6 in. or 2 ft or every time the type of soil changes, the number of tins required per mile of road is about 20-100; rather more are required per mile for a short survey than for a long one. The number of samples required depends on the soil conditions on the site and on the importance of the works proposed.

8·14 The type of hand-auger that has been found most generally useful is the post-hole auger of 4- or 5-in. diameter. On stony ground it may be supplemented by a crowbar or a gravel-auger, though the latter is seldom satisfactory. For very dry soil or ground below the water-table level an auger with self-closing flaps may be required, but this type is troublesome to use as the flaps are liable to jam. In cohesive soils free of stones a corkscrew auger may be used, but the hole produced is of rather small diameter, down which it is difficult to see when measuring the depth of the water-table. A typical hand-auger is shown in Plate 8·3A. These augers all need extensions for boring below about 4 ft, the extensions being made of ¾-in. gas-piping. For convenience, the auger itself and the extensions are made an exact number of feet in length, the auger usually being 4 ft long and the extensions 3 ft. The shaft of the auger and the extensions should be ringed at every foot to facilitate reading the depth at a glance.

TABLE 8·2

SIZE OF SOIL SAMPLE REQUIRED

Purpose of sample	Soil type	Minimum weight of sample required (lb.)
Soil identification and natural moisture content tests	Cohesive soils and sands Gravels	1½ 7
Compaction tests	Cohesive soils and sands Gravelly soils	10 20
Soil cement stabilization	Cohesive soils and sands Gravelly soils	50–100 100–200
Other methods of soil stabilization...	Cohesive soils, sands, and gravelly soils	10–20

8·15 For dealing with gravelly soils a power auger is useful. The Ministry of Transport own one such machine which is illustrated in Plate 8·3B. It is hoped to adapt this machine for taking undisturbed samples. This power auger is not of British manufacture and few are available in this country.

8·16 Boring records similar to those illustrated in Fig. 8·2 are required at the rate of five to thirty sheets per mile of road: this allows one sheet per hole and perhaps two sheets for those holes over 10 ft deep.

8·17 A list of equipment needed on a large-scale survey is given in Chapter 27. Where conditions make it impossible to carry out the work with the equipment described above, other methods of surveying may be used, details of which are given in paragraphs 8·33 and 8·34. The equipment for these methods is generally expensive and the work is almost always carried out by specialist firms.

BORING RECORD

Site....Greymouth............ Boring No...10....... Job No. I/1234/SR

Chainage..10 + 00.......... Date...10/2/43......

Natural Surface Level..124·6...... Existing Surface Level..124·6......

Formation Level....–.......... Water-table Level...118·1............

Remarks..In an old wood near the bottom of a steep bank. 1½-in. of dead leaves removed before boring.

Depth below surface ft	in.	Tin No.	M/C Tin No.	Natural Moisture Content %	Water seeping into hole	Soil Classn.	Remarks
0	0	136	104	10	—	SF	Dark sandy top soil
1	3	124	98	10	—	SF	Dirty brown cohesive sand
2	0	137	—	—	—	SF	Brown cohesive sand, wetter and plastic.
2	9	113	32	51	—	CH	Hard blue clay, plastic.
5	6	119	—	—	—	CH	Ditto, wetter
6	3	96	169	31	—	SP	Grey-brown mottled sand, very wet, fairly clean.
6	6				Water-table		
7	0	126	—	Water churned up with sample		SP	Clean green sand
7	6	—	—	"	Standing water, sides falling in	SP	Ditto.

FIG. 8·2 TYPICAL BORING RECORD SHEET

SOIL SURVEY PROCEDURE

8·18 The procedure in making a soil survey falls into four main parts:—

(1) Preliminary work.
(2) Site reconnaissance.
(3) Determination of the soil profile and collection of samples.
(4) Location of water-table.

Preliminary Work

8·19 Before proceeding to the site, the surveyor should first examine any existing information on the geological and soil conditions likely to be met. The 1-in. to 1-mile (and 6-in. to 1-mile if available) Geological Survey Maps and the appropriate Geological Survey Memoirs, particularly the sheet memoirs, will often give useful information. Both solid and drift maps should, if possible, be consulted. The data given include the nature of the underlying rock and superficial deposits, as well as the location of any known faults and igneous intrusions. It is also desirable to consult the Geological Survey and Museum as they often have useful unpublished information. In Great Britain soil maps exist for only a few areas, but for these areas they should provide useful additional information.

8·20 The centre-line of the road is set out on the 6-in. to 1-mile Ordnance Map or, if drainage investigations are planned, on the 1/2500 (25-in.) map. The lay-out of the borings should be plotted on one of these maps. If only a strip is to be surveyed, as is usual for a road, a single line of borings along the centre line or a double line offset 50 or 100 ft on each side, is usually sufficient, but if a wide area is to be surveyed the best way of covering the ground is probably with a grid. Normally the borings are required at intervals of about 300 ft. The depth of boring should be 4 to 5 ft below existing ground level or finished formation level, whichever is the lower, with an occasional deeper boring; for a high embankment, boring should be to a depth about equal to twice the height of the embankment if there is a possibility of soft material underlying it. The depth bored may be less in the valleys where it is obvious that some filling will be done and should be more on hills where cuttings will probably be required. From the geological information and lay-out of the borings more definite information can be obtained as to the total length of the borings and the number of sample tins required.

Site Reconnaissance

8·21 On first visiting the site it is desirable to make a preliminary reconnaissance by walking over the ground, to obtain a broad indication of the work required. When practicable, this site reconnaissance should be made before the main survey party proceeds to the site. Neighbouring quarries, cuttings and escarpments should be inspected. The general topography will often give some indication as to whether the soil conditions are likely to be variable or not. Some indication of the depth of the water-table below the surface can often be obtained from an inspection of such streams or ponds as may be seen, while vegetation such as rushes, cotton-grass or willows indicates that the water-table is near the surface for at least part of the year. Bracken and gorse are usually indications of a well drained soil. Silver birches and pines prefer a sandy soil, while beeches are usually found on clay.

8·22 A change in the vegetation over quite a small area may indicate an important change in the subsoil or rock formation, such as an igneous intrusion or the presence of alluvial soil. Plate 8·4A shows how an abrupt change in soil conditions is indicated by a change in vegetation: the growth of daisies on the bank in the foreground stops abruptly where marshy conditions occur in the lower ground in the middle distance. Plate 8·4B shows how wet soil conditions can affect the growth of vegetation; in winter the water-table rises above the bottom of the valley, which is consequently covered by water-loving plants and grasses, while in summer the water-table drops and the grass becomes parched and brown as shown in the photograph. Brown discoloration of surface water in swiftly flowing streams may denote the presence of peaty soil, though the peat may be outside the site. Geological faults may be indicated by topographical features such as a step in the line of an escarpment or a valley where the shattered rock has been more rapidly eroded than has the surrounding rock. These places should receive detailed inspection as igneous intrusions may involve rock excavation, while in certain circumstances a fault may also give rise to construction difficulties. Careful note should also be made, especially on sloping clay formations, of any irregularities in the ground that may be due to landslips or lack of stability. They may be indicated by a terraced appearance, and sometimes by the inclination of trees from the vertical.

8·23 After the site reconnaissance, the preliminary plan of the borings should be revised if necessary. The interval between the borings depends entirely on the nature of the soil encountered and may be as much as 1,000 ft in very uniform ground, or as little as 50 ft in quickly changing ground such as glacial deposits.

Determination of Soil Profile

8·24 SETTING OUT. When the lay-out of the borings has been decided the position of the holes should be pegged on the ground. Except on moorlands or in very large fields, it is usually sufficiently accurate to set out the positions of the holes by setting ranging rods or other suitable markers in the hedges on the line of the borings. Only in exceptional cases will the use of a theodolite be necessary.

8·25 BORING. As previously mentioned, the post-hole auger is the most generally useful tool for hand-boring. In gravelly soils the gravel auger may be used, but if there are many large stones even this may prove unusable, and it is then best to use the post-hole auger in conjunction with a crowbar. The stones are removed singly by using the crowbar to push them into the lowered auger which has a little soil worked into it to act as a cushion. When the auger is being raised, it may be necessary to hold the stone in position with the crowbar, until it can be reached by hand and thrown out. If these methods fail, it may be necessary to start another hole nearby or to dig down to the obstruction if this is near the surface. If the soil is too dry to be retained in the auger, and no auger with self-closing flaps is available, water may be poured down the hole, but under these conditions samples cannot be taken for moisture content determinations for several feet below the depth at which this procedure was employed. The introduction of water is not recommended except as a last resort. When boring in areas where field drains have been constructed, there is sometimes a danger of mistaking the gravel and stone frequently used for surrounding the pipes for changes in the soil strata.

8·26 Beds of peat or soft clay should be completely penetrated and their boundaries located. When rock is encountered it is unnecessary to proceed further, since rock is a satisfactory foundation, but in glaciated areas care should be taken not to confuse boulders with solid rock.

8·27 SAMPLING. At least one sample should be taken from each stratum found in each boring. Samples should be placed in separate sample tins and their nature noted on the boring record. Reliable and experienced labourers can be left to do this on their own, but others may have to be closely supervised unless some such plan as the following is adopted:—Close to the proposed hole, a strip of ground should be cleared and roughly smoothed over, then as the first auger-full of soil is brought up it is laid at one end of this strip, the next load is laid adjacent to it and so on. As each foot of depth is passed a deep scratch is made beside the last auger-load of soil that was brought up. In this way, the engineer can see exactly what soil has been brought up from each part of the hole and can select the samples that he requires. The size of samples necessary for various purposes has already been given in Table 8·2.

8·28 RECORDS. Full and systematic records must be kept of each boring on boring record sheets (Fig. 8·2). Part of the boring record is filled in on the field and part subsequently. The description of the site should give the nature of the ground, e.g. a ploughed field, and should state the depth (if any) of loose leaves or sods removed before boring. The description of the soil should include its colour, texture, consistence, structure, stoniness and a statement as to whether it is highly organic or not. In clays, a mottled colour implies poor drainage, while a blue colour indicates that no oxidation has taken place. Under the heading " texture," a note should be made of the coarseness or fineness of the material; the consistence can be described as hard, stiff, soft, etc. The soil structure will be disturbed by the auger but it may sometimes be discernible from the larger lumps of soil and may be described, for example, as granular or laminar; the presence of organic matter may be shown by the dark colour and the smell of soils containing it. If the soil is so stony that a reasonably representative sample cannot be obtained, a note should be made of the approximate proportion of stone in the soil and the range of sizes found. Chapter 4 contains a more complete selection of descriptive terms useful in differentiating soils in the field. A tentative designation of each soil according to a suitable classification system (see Chapter 4) may also be usefully included in the boring record at this stage.

8·29 GRAVELLY SOILS. In gravelly soils progress with a hand auger will be subject to frequent delays even when a crowbar is also used and it may become necessary to resort to other methods of exploration. The ordinary well-drilling equipment is almost useless for this work as the only sample recovered is in the form of slurry which is unsuitable for soil testing. In cases of this sort, a power auger such as is shown in Plate 8·3B is useful. The machine shown is mounted on a four-wheel-drive lorry and weighs about 9 tons. It can drill holes of from 12-in. to 48-in. diameter, to a depth of 20 ft. The larger drills are useful when undisturbed samples have to be obtained from a considerable depth as they make a hole large enough to enable someone to go down the hole to obtain the samples, if the soil conditions are such that this is safe.

Location of Water-table

8·30 Should no trace of water be found in the hole at its full depth, it may be backfilled at once, but if water is found or indicated, the hole should be left for 12 to 24 hours for the water to rise to its final level. The most convenient way of measuring the depth to the water level is to lower a levelling staff or a ranging rod until its lower end just touches the water surface. For depths too great for this method to be used, a weighted tape can be used.

8·31 It occasionally happens that a boring pierces a layer of impervious clay resting on a permeable layer in which the water is under pressure, i.e. there are artesian conditions. These conditions will be shown by the water rising above the level at which it was first found seeping into the hole, and often indicate that special drainage will be required. Under other conditions, the boring may disturb a zone of more than usually permeable soil in which a very rapid flow is taking place. In both these circumstances it would be well to put down another hole nearby as a check. It is a good plan to plot the information obtained while still on the site to avoid returning to the base with insufficient data.

Geophysical Methods

8·32 Where speed is more important than the accuracy of the information about the types of soil present, geophysical methods can sometimes be used to locate the approximate depth of strata or the water-table, but specialists must be employed for the work. Both the methods described in the following paragraph are widely used in prospecting but, as far as is known, only the resistivity method has been used on road projects in this country. The seismic method is generally used where information concerning soils or rocks at a considerable depth is required while the resistivity method can be used for shallow depths as well. Both methods have been used to determine depths to solid rock or to the water-table, even when these were shallow.

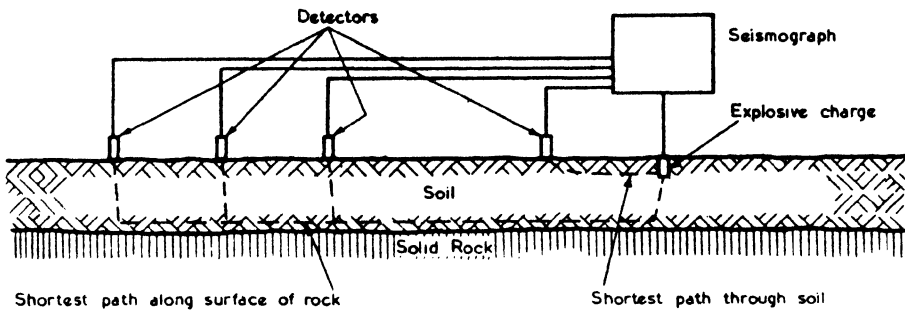
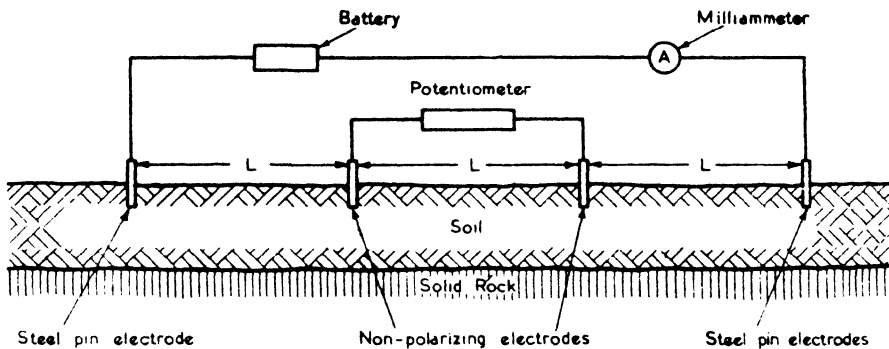
8·33 SEISMIC METHOD. The seismic method depends on the fact that sound and shock waves travel faster in rock than in soil and faster in solid rock than in fissured rock or soil. Fig. 8·3a illustrates how a shock wave may travel more quickly by penetrating deeply to a stratum of solid rock than by travelling through the soil above it, provided that the horizontal distance to be travelled is substantially more than twice the thickness of the soil. The shock that first operates the farther detectors is one which travels down through the soil at such an angle that it is refracted along the surface of the bed-rock below it, and in travelling along this surface is constantly refracted upwards as shown. The speed of the shock wave varies from 1,000 to 20,000 ft/sec. By recording the times taken for the shock wave to be transmitted, both directly through the soil and *via* the surface of the solid rock, from the firing point to each of several detector instruments, the seismograph readings enable the interpreter to determine the depth to solid rock. Ground below the water-table behaves similarly to rock, in that it transmits a shock wave at high speed. It is clear from the foregoing that to gain more than a sketchy idea of subsurface conditions, it is desirable to supplement the geophysical measurement with a few borings.

(a) Seismic method

(b) Resistivity method

FIG. 8·3 GEOPHYSICAL METHODS OF SURVEYING

8·34 RESISTIVITY METHODS. Of the several electrical methods of surveying, the one most used in road work is the resistivity method illustrated in Fig. 8·3b. Two electrodes are set in the ground, their distance apart being of the order of three times the depth to which information is required. Two more electrodes of a non-polarizing type are set in the ground between and in line with the first two, usually at the third points. A current is passed through the ground *via* the outer two electrodes; this may be from a battery, but in districts where electricity is in use it is common to generate an alternating current with a square wave form. In this case the current can be distinguished from any stray currents in the soil and it is not necessary to use non-polarizing electrodes. Part of the current is picked up by the intermediate electrodes and the resistance of the circuit measured by the potentiometer. By increasing the spacing of these electrodes the corresponding depth to which measurements of resistance are taken is increased, and in general the spacing of the central electrodes at which a marked change of resistance is indicated is equal to the depth of the stratum causing it. A list of books for further reading on geophysical surveys is given at the end of this chapter.

THE APPLICATION OF SOIL CLASSIFICATION TO SITE INVESTIGATIONS

8·35 The information obtained from the field work must be reduced to a form in which it can be applied to engineering problems. To permit deductions to be made from a knowledge of the type of soil it is desirable to classify the soil according to some recognized scheme, such as is described in Chapter 4.

Selection of Samples for Classification

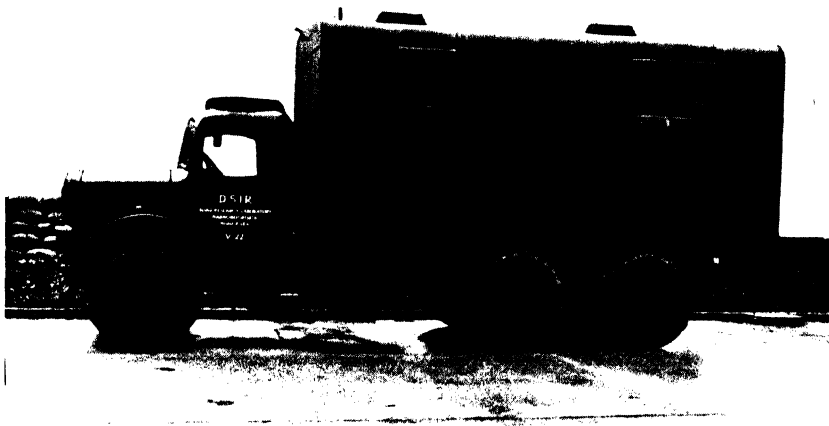
8·36 The first step in classifying the soils from any site is to set out the samples in some logical order, e.g. the samples from each boring can be set in one row with the top sample at the back and the next in front of it, and so on. The soils can then be grouped into a limited number of types, from each of which a few samples representing the extremes and means of particle-size distribution and consistence should be selected for classification tests. The grouping is at this stage carried out by means of visual comparison of one soil with another, note being taken of the field description of each soil and of any tentative grouping for classification purposes suggested by the site. Reference should also be made to the relative positions of the soils on the site, since soils of one type tend to occur in continuous beds.

Soil Survey Classification

8·37 The classification recommended for use in soil surveys for roads and other forms of construction with shallow foundations is the Casagrande system, described in Chapter 4. This gives good correlation with field observation but it may sometimes be desirable to limit the number of soil types in order to maintain a simple and workable scheme for use on the site. It is often possible to devise on any given site a more detailed *ad hoc* classification which may have advantages for certain purposes.

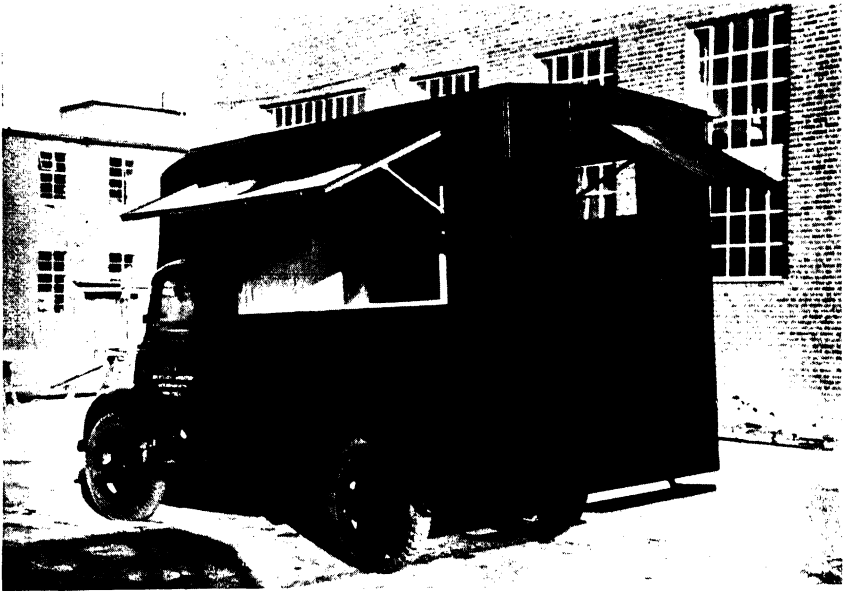
8·38 As an example, visual examination was used in the course of an extensive soil survey undertaken to discover the sites for borrow pits that would yield the best sand for making sand asphalt by the wet-sand process. Fig. 8·4 represents part of the soil profile mapped from the results of the classification. The five types of sand varied sufficiently in clay content to make certain varieties entirely unusable, and certain others unusable in wet weather. The behaviour of the materials when mixed with bitumen was determined in the laboratory and confirmed on the site. Only types IV and V were usable throughout the year; type III was satisfactory in the summer. The desirable sands were therefore types IV and V, and the most favourable sites for the borrow pits are obvious from inspection of the profile.

Interpretation of Results

8·39 The interpretation of the results of a soil survey depends largely upon the purpose of the investigation and the location of any proposed construction. The classification of soils encountered in a soil survey can only be interpreted as giving the general soil characteristics and conditions likely to be found. Table 4·1 indicates these general properties of soils as inferred from a classification of soil types and Tables 8·3 and 8·4 suggest typical slopes of cuttings and allowable bearing pressures for various soils.

(a) Exterior

(b) Interior

MOBILE LABORATORY FOR SOIL TESTING
used for complete soil investigations

PLATE 8·1

(a) Exterior

(b) Interior

SMALL MOBILE LABORATORY

for soil survey work and control tests

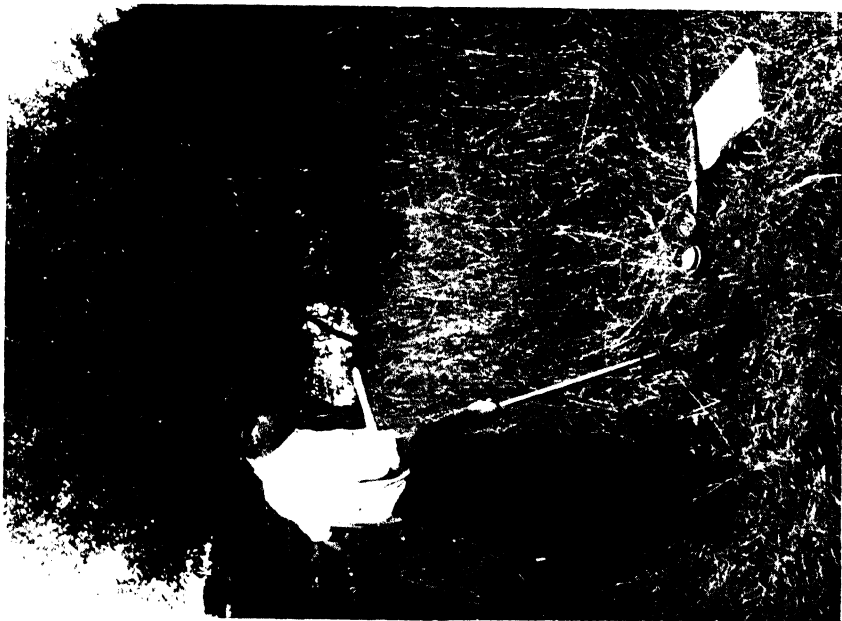
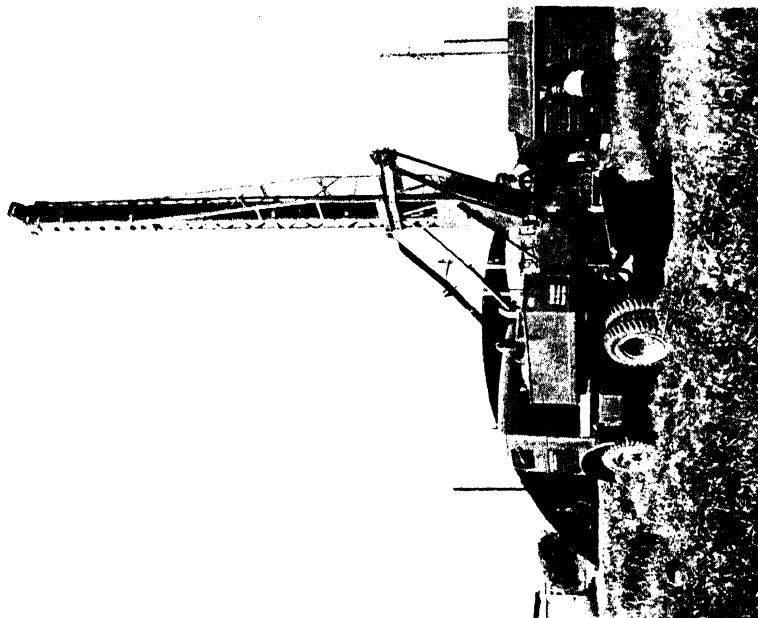
PLATE 8·2

(A) POST-HOLE AUGER AND CROW-BAR

(B) POWER AUGER SUITABLE FOR BORING IN GRAVELLY SOILS

PLATE 8·3

(A) CHANGE IN SOIL CONDITIONS INDICATED BY CHANGE IN VEGETATION

(B) HIGH WINTER WATER-TABLE INDICATED BY PARCHED WATER-LOVING GRASSES IN SUMMER

PLATE 8·4

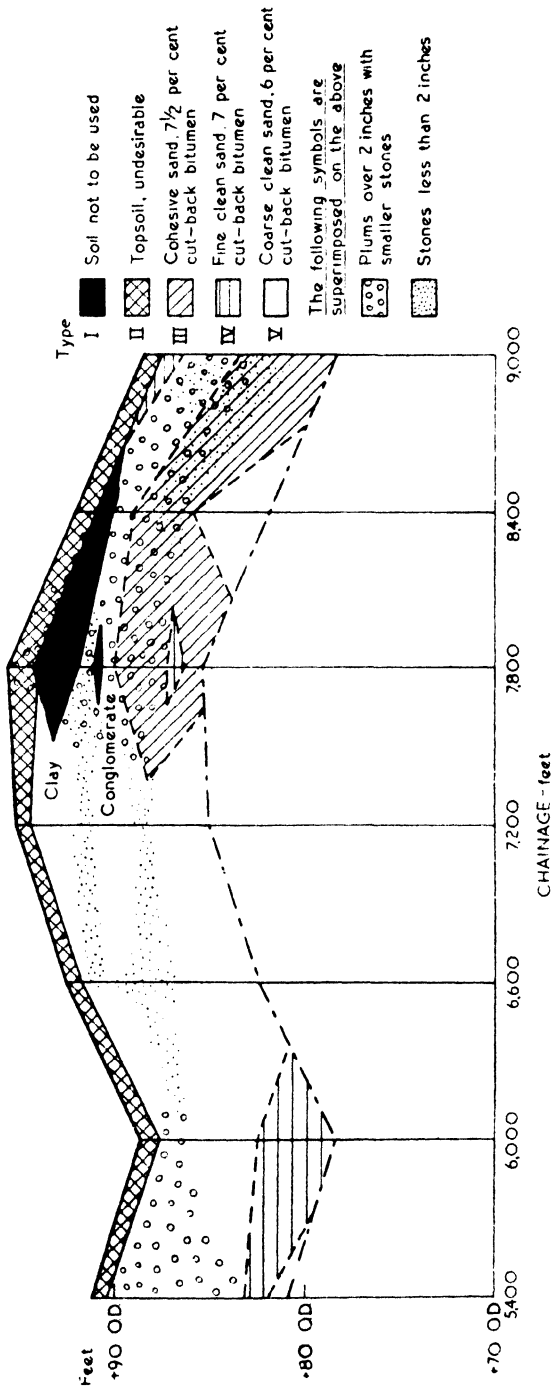



FIG. 8·4 SOIL PROFILE ILLUSTRATING USE OF VISUAL CLASSIFICATION AS APPLIED TO SANDS FOR BITUMINOUS SURFACING

8·40 More often, however, the soil survey and subsequent classification is used in conjunction with some special investigation such as measurements of soil strength or density, and tests for stability, tests for estimating settlements, tests for examining the possibility of stabilization or tests for the design of pavements. The classification plays a part subsidiary to these direct investigations and is often used merely as a method of selecting samples for tests.

GROUND-WATER INVESTIGATIONS

8·41 The results of ground-water investigations are recorded as:—

(1) Water-table shown on the soil profile.
(2) Water-table contours over the site.
(3) Lines of equal depths to water-table below the surface.

8·42 The water-table contours give the general direction of flow of the ground-water and thus indicate the points where, if necessary, this flow is best intercepted. Areas where the ground-water is near the surface are generally in need of subsoil drainage. Water-table investigations are generally not practicable in impervious clay soils in which the free water is present in the fissures in the clay and no true water-table exists.

8·43 For ground-water investigations a 25-in. to 1-mile or larger-scale plan is desirable. If information is to be given on the depths to the water-table, ground contours at 1-ft intervals are required.

TABLE 8·3

TYPICAL SLOPES IN CUTTINGS

Type of ground	Typical safe slope on excavations
Igneous rocks in sound condition	Almost vertical
Slates, schists, hard shales, hard chalk	¼:1
Thinly bedded limestones and sandstones, mudstones ...	¾:1
Clay-shales, friable sandstones	1½:1
Gravel and sand	2 :1
Fine sands	2½:1
Clay and silts	Depends on strength of material and depth of excavation

Note.—The slopes given in this table apply only to strata with more or less horizontal bedding planes. It must also be emphasized that the safe slopes given are only approximate typical values. Individual cases may deviate considerably from the values quoted.

8·44 To determine the water-table contours, the first step is to plot on the plan the positions of the borings with the level, reduced to Ordnance datum, of the water-table beside each point. The water-table contours can then be drawn by connecting points with equal levels. The resulting contours are usually even in shape and show few sudden breaks or irregularities: not infrequently they follow the general lie of the ground although exceptions do occur. Thus a rise in ground level is frequently associated with a rise in the water-table level. Springs, however, will produce marked changes in the shape of the contours.

TABLE 8·4

TYPICAL WORKING BEARING PRESSURES FOR STRUCTURAL FOUNDATIONS

Material	Typical working bearing pressure (tons/sq.ft)
Non-cohesive	
Compact gravel or sand and gravel	5
Loose gravel or sand and gravel	3
Compact coarse sand	4
Loose coarse sand	2
Compact fine sand	3
Loose fine sand	1
Cohesive	
Very stiff boulder clays, clay shales	6
Stiff clays and sandy clays...	4
Firm clays and sandy clays	2
Soft clays and silts	1
Very soft clays and silts and peat...	not more than ½
Hoggin (compact)	6
Hard solid chalk	6

Note.—The above values are only approximate and the allowable bearing pressure for individual soils may differ considerably from the values quoted. It is always preferable and often essential to obtain the bearing capacity of soils directly from strength measurements.

8·45 To obtain lines of equal depth to ground-water, the ground-water contours are superposed on ground-level contours as in Fig. 8·5. The intersections of the two families of contours form a series of diamond-shaped areas, as will be seen on the right-hand side of Fig. 8·5. From the intersections of the two families of curves, points are selected where the differences between ground and water-table levels are constant. For example, take X in the figure where this difference is 2 ft., and join to the nearest intersection where the interval is also 2 ft, i.e. at Y. Continuing in this manner, the line where the ground-water is 2 ft below surface is gradually developed. The points joined mostly occur diagonally opposite across the diamonds, but care must be taken to see that the correct corners are joined. In general, these lines of constant depth to water-table form irregular closed loops, but frequently the closure is off the edge of the plan concerned. To avoid confusion it will be found convenient to draw each set of contours in a different colour and in the report to give the lines of equal depth to ground-water on a separate drawing.

8·46 It is also possible, of course, to plot the depths to the ground-water directly from the depths of water in the borings. This method takes no account of the intermediate surface irregularities unless the bor ngs are very numerous or the ground very flat. In practice it is found that the first method affords the easiest way of obtaining the contours of the ground-water.

8·47 The actual levels of the ground-water naturally vary with the season, and for this reason it is desirable to carry out the survey when the water-table

is at its highest; but whatever the time of year the directions of flow will usually be the same, and the areas where the water is nearest the surface will usually be those at which it stands closest to the surface in winter. Where the water is at ground level, drainage is obviously necessary, but where the water-table is below ground, it is necessary to estimate the height to which it may rise to fix the area that should be drained. An example of the value of such information about the ground-water can be illustrated by the case of a fly-over crossing of the existing Bath Road by a proposed new arterial road, east of Slough (see Fig. 8·6). A tentative scheme was drawn up in which the existing road was raised a few feet and the new road passed underneath it. In the course of a soil survey of the new road, it was found that the summer (1946) water-table was about 8 ft above the proposed level of the new road at this crossing and that the soil was Thames ballast. This indicated that the scheme would involve heavy expenditure on drainage works, even if it proved practicable, and a second scheme with approximately the same plan was then drawn up in which the existing road was dropped to just above winter water-table level and the new road carried over it.

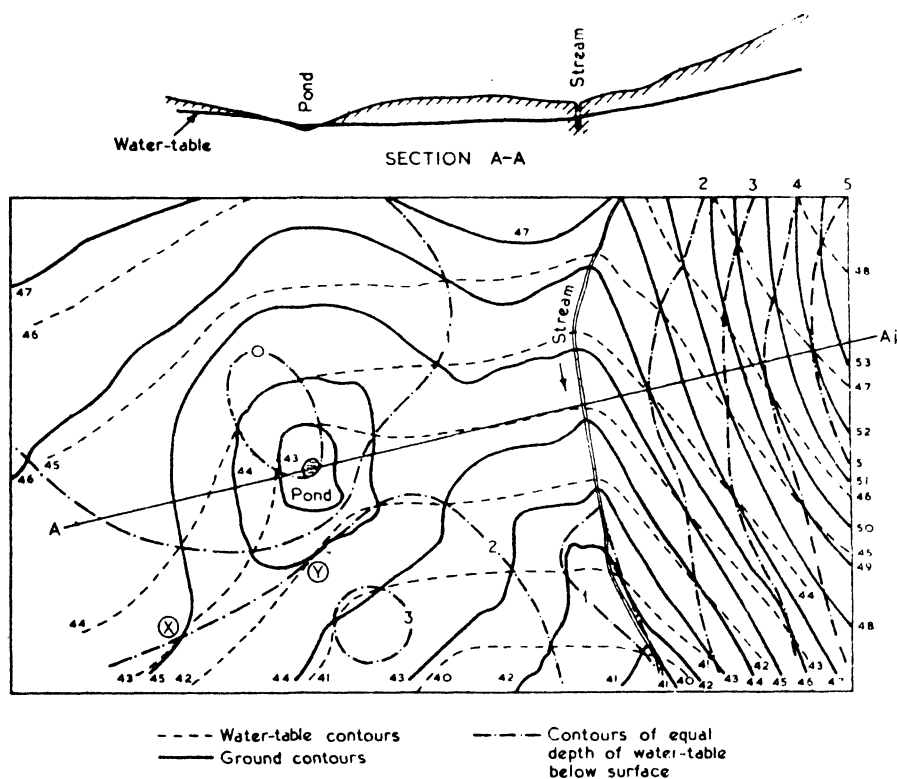



FIG. 8·5 PLAN SHOWING GROUND AND WATER-TABLE CONTOURS AND CONTOURS OF EQUAL DEPTH OF WATER-TABLE BELOW SURFACE

(a) First scheme

(b) Second scheme

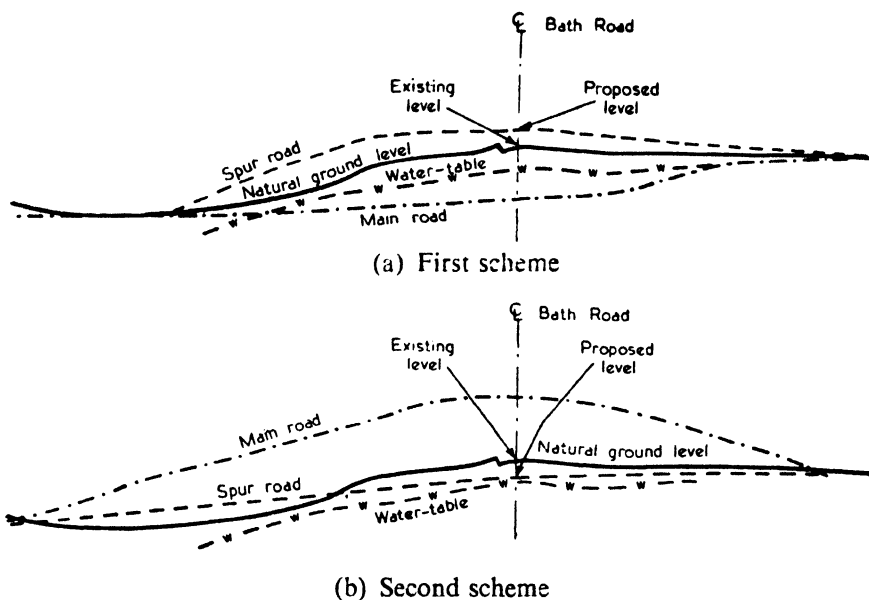



FIG. 8·6 LONGITUDINAL SECTION ALONG PROPOSED NEW ROAD

PRESENTATION OF INFORMATION

Soil Profile and Contract Drawings

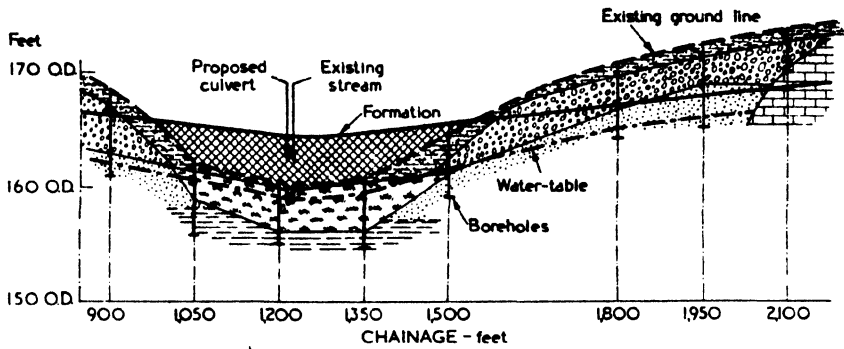
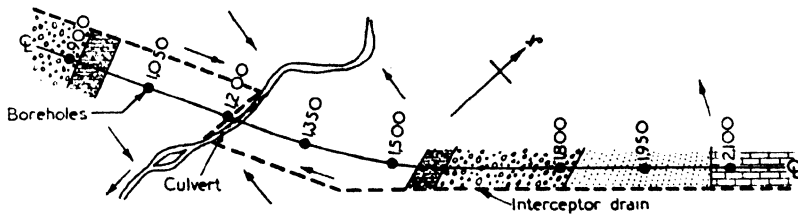
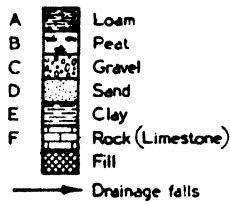
8·48 Recording the information obtained from the borings in suitable form for reports and contract drawings involves drawing a soil profile. For the purpose of a report, it is sometimes advantageous to give the soil profiles, plans and other information on separate sheets, but frequently it may be preferable to incorporate as much information as possible on a single drawing which can then be regarded as a contract drawing. The information which should be included on this drawing comprises:—(1) soil profile, (2) plans, (3) typical sections, (4) key to soil types and (5) table of test results on soils. Table 8·5 indicates in more detail the information that should be included and the scales recommended. Fig. 8·7 illustrates a drawing of this type. Considerations of the scale of reproduction make it necessary in this diagram to show only a very short section of profile but in other respects the drawing is complete.

Preparation of Report

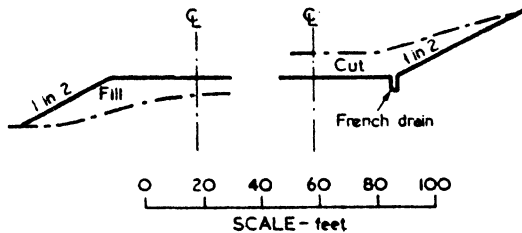
8·49 It is frequently necessary to present the information obtained in the form of a report, the nature of which depends very much upon circumstances. Reference here will therefore be confined to essential features. The report should begin with a general description of the topography of the site and of the soils encountered with reference to relevant geological information. A plan of the site, on a scale of 6 in. to 1 mile, should be included to indicate the general disposition of the proposed works in relation to the borings taken. As already indicated, the principal information obtained may be presented either on a single drawing which may ultimately serve as a contract drawing or on a number of separate sheets. Salient features of these diagrams should receive detailed mention.

TABLE 8·5

INFORMATION REQUIRED ON SOIL-PROFILE CONTRACT DRAWINGS

Item No.	Item	Scales of drawing	Information to be shown
1	Soil profile	Horizontal: 1/2,500 or 6 in. to 1 mile Vertical: 1 in. to 5-10 ft depending on topography and detail to be shown	1. Profile of existing ground surface and of proposed formation level, normally along centre-line of road. 2. Chainages. 3. Datum for levels. 4. Extent of different soils referred to key to soil types (see Item No. 4.) 5. Notes on soil types. 6. Proposed drains. 7. General notes and recommendations.
2	Plan	Horizontal: 1/2,500	1. Outcrops of different soil types at formation level. 2. Direction of falls of existing ground level. 3. Proposed drains.
3	Typical sections	Horizontal } Vertical } $\frac{1}{8}$ in. to 1 ft	1. Surface and subsoil drains. 2. Maximum slopes of cuttings and embankments.
4	Key to soil types	—	To present interpretation of soil profile (Item 1) and to be referred to Item 5.
5	Table of test results on soils	—	Summary of test results (if any) on different types of soil.

PROFILE ALONG CENTRE LINE OF ROAD

PLAN SHOWING SOIL TYPES EXPOSED AT FORMATION LINE ON CENTRE LINE AND DRAINAGE LAYOUT

SOIL TYPE	C	D	E
Gravel (> 2 mm)%	60	41	0
Sand (2·0–0·06 mm)%	39	55	41
Silt (0·06–0·002 mm)%	1	4	26
Clay (< 0·002 mm)%	0	0	33
Liquid limit	NP	NP	45
Plastic limit	NP	NP	20
Plasticity index	NP	NP	25

NP = Non-plastic

TESTS ON CHIEF TYPES OF SOIL

℄ ℄
1 in 2 Fill
Cut 1 in 2
French drain
0 20 40 60 80 100
SCALE - feet

TYPICAL CROSS-SECTIONS

NOTES:

All topsoil (Type A) is to be preserved and used for soiling at the completion of the work.
All peaty soil (Type B) is to be cut out and replaced by approved filling.
All filling is to be spread and rolled in layers not exceeding 9 inches.
Sand is to be spread 2 inches thick as sub-base.

FIG. 8·7 TYPICAL SOIL SURVEY DRAWING SHOWING ESSENTIAL REQUIREMENTS FOR CONTRACT DRAWING

TABLE 8·6

EXAMPLE OF CONCISE METHOD OF REPORTING THE RESULTS OF SOIL TESTS ON TYPICAL SAMPLES

Reference	a.	b.	c.	d.	e.
Test information	Topsoil	Clean sand	Silty sand	Sandy clay	Silty clay
No. of samples tested	2	3	4	3	3
Particle-size distribution					
Passing No. 7 B.S. sieve (2·0 mm.)... ...	100	95–100	92–100	98–100	98–100
Passing No. 25 B.S. sieve	76–78	68–90	64–87	72–89	70–98
Passing No. 72 B.S. sieve	58–59	35–61	40–56	47–65	59–95
Passing No. 200 B.S. sieve (0·06 mm.) ...	40–42	10–41	19–38	22–50	55–69
Finer than 0·02 mm....	29–31	—	12–38	14–38	42–50
Finer than 0·002 mm.	11–21	—	9–18	10–27	29–45
Gravel: (>2·0 mm.)	0	0–5	0–8	0–2	0–2
Sand: (2·0-0·06 mm.)...	58–60	59–87	55–90	49–78	31–43
Silt: (0·06-0·002 mm.)	21–29	10–41	10–20	13–23	10–40
Clay: (<0·002 mm.)	11–21	nil	9–18	10–27	29–45
Soluble in H_2O_2 (organic matter) (%) ...	0–8	0	0	0	0
Soluble in HCl (carbonates) (%)	0–4	0–11	2–13	8–19	3–8
Moisture content (above water-table) (%) ...	13–56	8–15	16–24	21–48	34–55
Liquid limit	36	22*	19*	22–32	46–55
Plastic limit	27	—*	13*	12–17	21–25
Plasticity index	9	—*	4*	10–16	25–34
Soil classification	ML	SF	SF	CL	CH

*In some samples the soil contained too much sand to permit test results to be obtained.

8·50 The ground-water investigation (if any) should be described, although the practical inferences will have been embodied on the contract drawings. Reference should be made, if necessary, to the results of special investigations. A summary of the principal recommendations should be set out at the head of the report. These should cover as far as is possible all the appropriate items set out in Table 8·1. Unless included in the drawings, the results of identification tests can be given conveniently in tabular form at the end of the report, as indicated in Table 8·6.

SUMMARY

8·51 The purpose of a soil survey is the exploration of soil and ground-water conditions along the line of a proposed road. Soils occurring in various strata below the surface of the ground are located by sinking borings usually with hand augers. The water-table is determined by allowing water in the bore holes to reach equilibrium. The procedure for making a soil survey is described, and examples of its use are quoted. Details are given of the personnel and equipment required for a survey and reference is made to tests necessary to identify and classify soils and also for tests to be made in connexion with such problems as the stability and settlement of foundations. Seismic and electrical resistivity methods that have been intensively used in the U.S.A. are described; these methods are useful where it is necessary to determine rapidly depths of solid rock or water-table.

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CHAPTER 9

COMPACTION OF SOIL

INTRODUCTION

9·1 Soil compaction is the process whereby soil particles are constrained to pack more closely together through a reduction in the air voids, generally by mechanical means. In recent years, considerable advances have been made in the knowledge of the subject, and this has enabled better results to be obtained with equipment used for compacting soils in the field.

9·2 By compacting soil under controlled conditions, the air voids can be almost eliminated and the soil can be brought to a condition in which there will be less tendency for subsequent changes in moisture content to take place. In a well compacted embankment, the settlement of the fill is negligible, although there may be consolidation of the soil on which the embankment is founded. Thus, when an embankment is well compacted, the practice of allowing it to " weather " may be dispensed with, and the pavement constructed immediately on completion of the earthworks. Well compacted bases and subgrades also possess high strength and resistance to deformation.

STUDY OF SOIL COMPACTION IN THE LABORATORY

Measurement of Soil Compaction

9·3 Compaction is measured quantitively in terms of the dry density of the soil, which is the weight of soil solids per cubic foot of the soil in bulk. The moisture content of the soil is the weight of moisture present expressed as a percentage of the weight of dry soil, and the dry density is thus determined from the bulk density of the soil by deducting the weight of moisture present.

Factors influencing the Compaction of Soil

9·4 The increase in the dry density of soil produced by compaction depends mainly on the moisture content of the soil and on the amount of compaction applied. With a given amount of compaction there exists for each soil, as shown in Fig. 9·1, a moisture content termed the " optimum moisture content " at which a maximum dry density is obtained.

9·5 With a soil of given moisture content, increasing amounts of compaction result in closer packing of the soil particles and in increased dry density, until the volume of air remaining in the soil is so reduced that further compaction produces no substantial change in volume. The effects of the principal variables will be considered in detail.

9·6 MOISTURE CONTENT. The behaviour of the soil at different moisture contents can be explained as follows. When the moisture content is low, the soil is stiff and difficult to compress; thus, low dry densities and high air

contents are obtained (see Fig. 9·1). As the moisture content increases, the water acts as a lubricant, causing the soil to soften and become more workable. This results in higher dry densities and lower air contents. As the air content becomes less, the water and air in combination tend to keep the particles apart, and prevent any appreciable decrease in air content. The total voids, however, continue to increase with the moisture content, and hence the dry density of the soil falls.

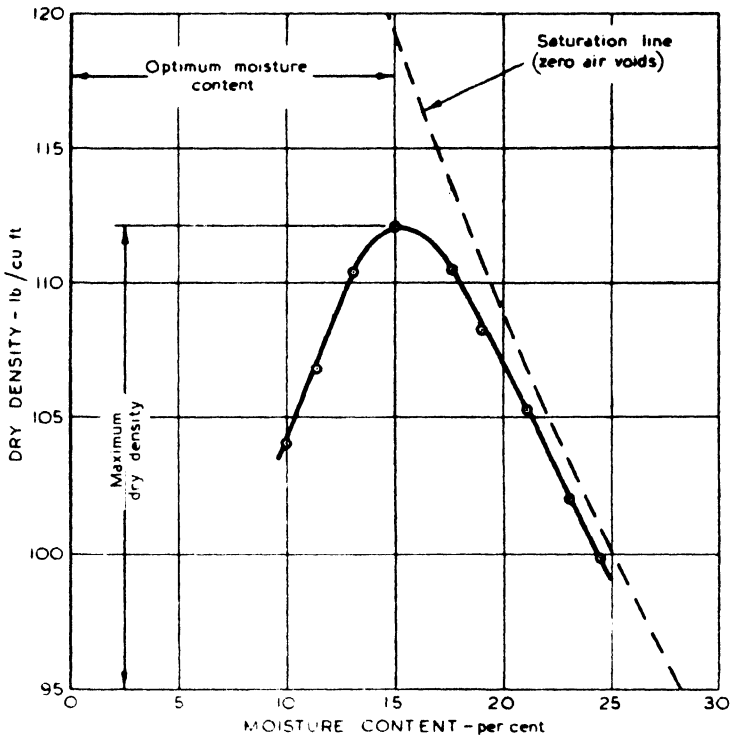



FIG. 9·1 RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR CONSTANT AMOUNT OF COMPACTION

9·7 To the right of the peak of the dry density/moisture content curve (Fig. 9·1) the saturation line (the theoretical curve relating dry density with moisture content for soil containing no air voids) is approached but never reached, since it is never possible to expel by compaction all the air entrapped in the voids of the soil.

9·8 Amount of compaction. For all types of soil and with all methods of compaction, increasing amounts of compaction, i.e. the energy applied per unit weight of soil, result in an increase in the maximum dry density and a decrease in the optimum moisture content.

9·9 Fig. 9·2 shows the effects of increased amounts of ramming on a sandy clay soil (liquid limit 29 per cent, plasticity index 7 per cent). The lower curve shows the dry density/moisture content relation for the soil compacted by

applying 25 blows per layer on each of three layers according to the procedure for the B.S. compaction test (see later), and the upper curve the corresponding relationship when 100 blows per layer were applied. Comparing these curves, it will be seen that above the optimum moisture content, when the percentage of air voids was small, increased compaction had little effect upon the dry density, but below the optimum moisture content, when the air voids were large, the effect of increased compaction was considerable.

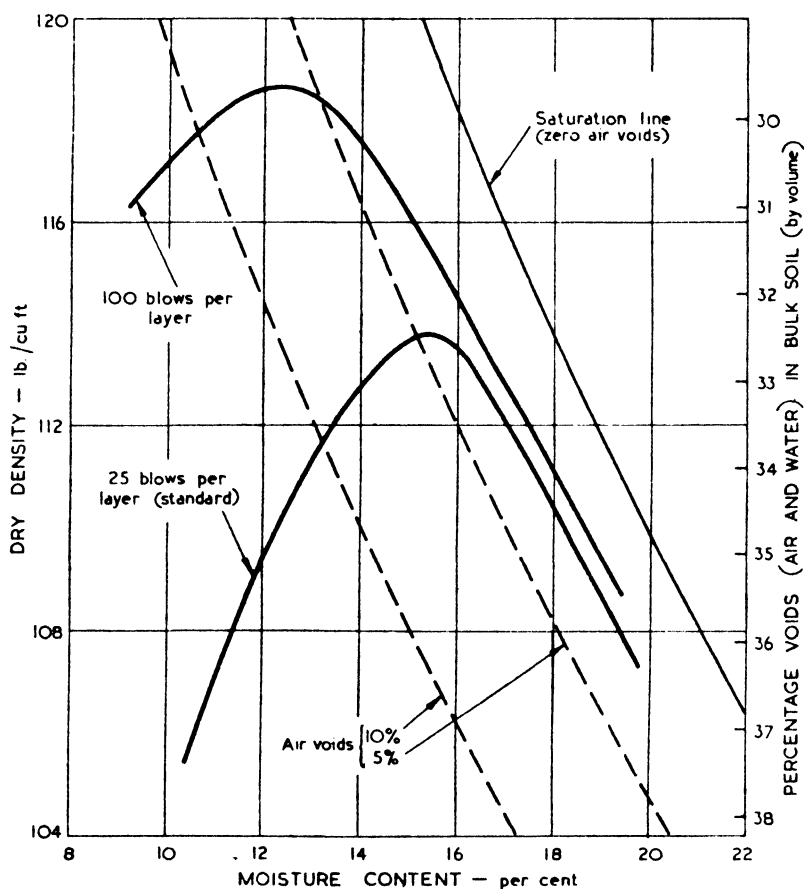



FIG. 9·2 EFFECT OF DIFFERENT AMOUNTS OF COMPACTION ON DRY DENSITY OF SANDY CLAY SOIL

9·10 The same effect occurs when soil is compacted under static pressure. Data given by Hogentogler[1] have been plotted in Fig. 9·3 using a logarithmic scale of moulding pressures. Over the range of pressures examined, linear relations were obtained between the logarithm of the moulding pressure and both the maximum dry density and the optimum moisture content (by volume). Had the range of pressures been extended, the dry density must have approached some limiting value.

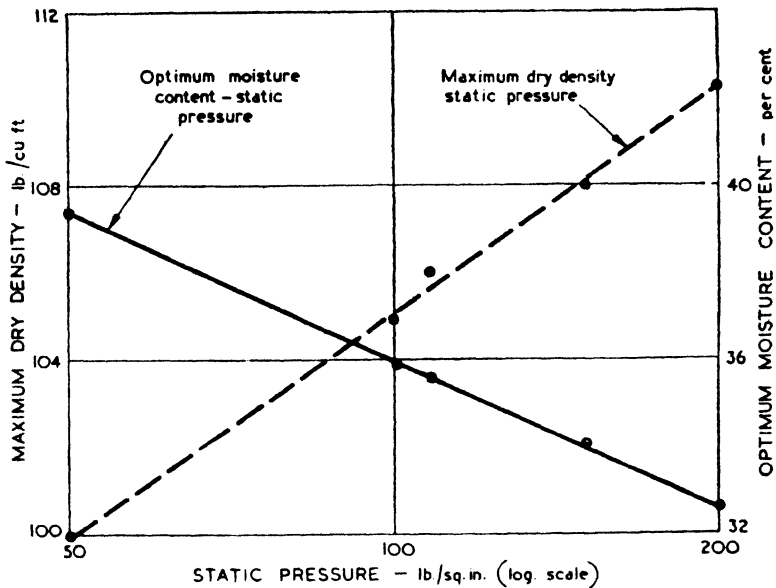



FIG. 9·3 EFFECT OF STATIC PRESSURE ON MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT (BY VOLUME) OF SOIL
Data by Hogentogler

9·11 SOIL TYPE. The maximum dry density that can be obtained either in laboratory tests or in the field with any soil depends upon its type and varies from about 140 lb./cu.ft for a well graded gravel to about 90 lb./cu.ft for a heavy clay. The optimum moisture content of British soils ranges from about 4 per cent for coarse-grained soils to about 28 per cent for heavy clays. Typical dry density/moisture content curves obtained with a laboratory compaction test for a wide range of soils are shown in Fig. 9·4. These curves were obtained by K. B. Woods[2] by classifying and averaging data on 1,383 Ohio soils. Curves of constant air voids for a soil composed of particles with a specific gravity 2·70 have been superposed on Woods' curves. It will be seen that on the right-hand side all the curves approach the saturation line and the peaks all occur at an air voids content of approximately 5 per cent. The average curves are all rather similar in shape. Generally speaking, a flat curve denotes a closely graded soil, and a curve with a pronounced peak denotes a well graded soil. This is illustrated in Fig. 9·5 where laboratory dry density/moisture content relations are compared for two cohesionless sands.

9·12 Other factors affecting the compaction of soil are those of temperature and admixtures (see Chapters 10 and 12).

Laboratory Tests used in connexion with the Compaction of Soil

9·13 DETERMINATION OF THE ZERO AIR VOIDS LINE. The zero air voids line is readily determined from a knowledge of the density of the soil (γ_s). This can be determined by the standard laboratory method (B.S. 1377:1948, Test No. 7A) and in the field by a similar method employing a pycnometer (B.S. 1377: 1948, Test No. 7B) (see Chapter 3).

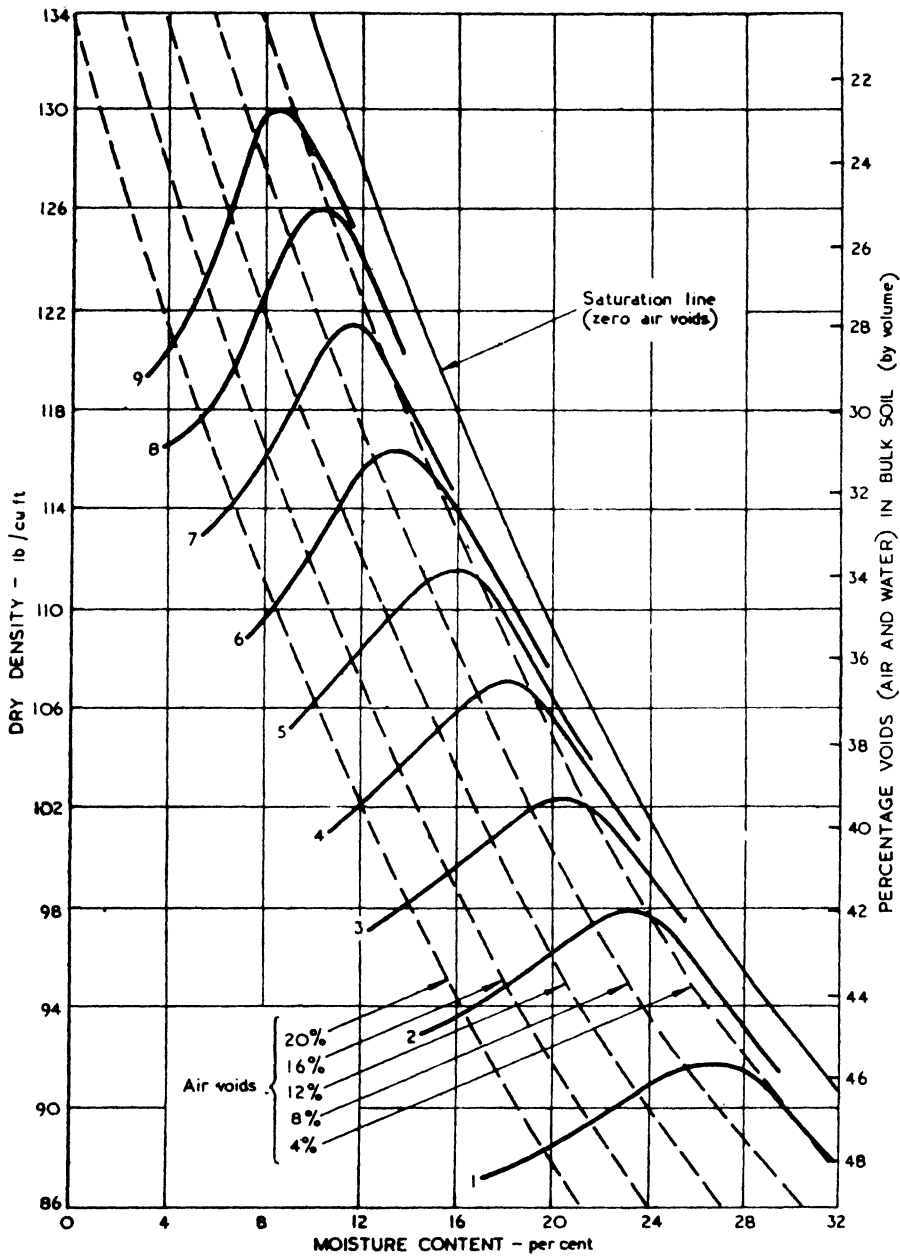



FIG. 9·4 AVERAGE RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR SOILS HAVING MAXIMUM DRY DENSITIES DIFFERING BY 5 LB./CU.FT

Based on experimental data by K. B. Woods covering 1,383 soils

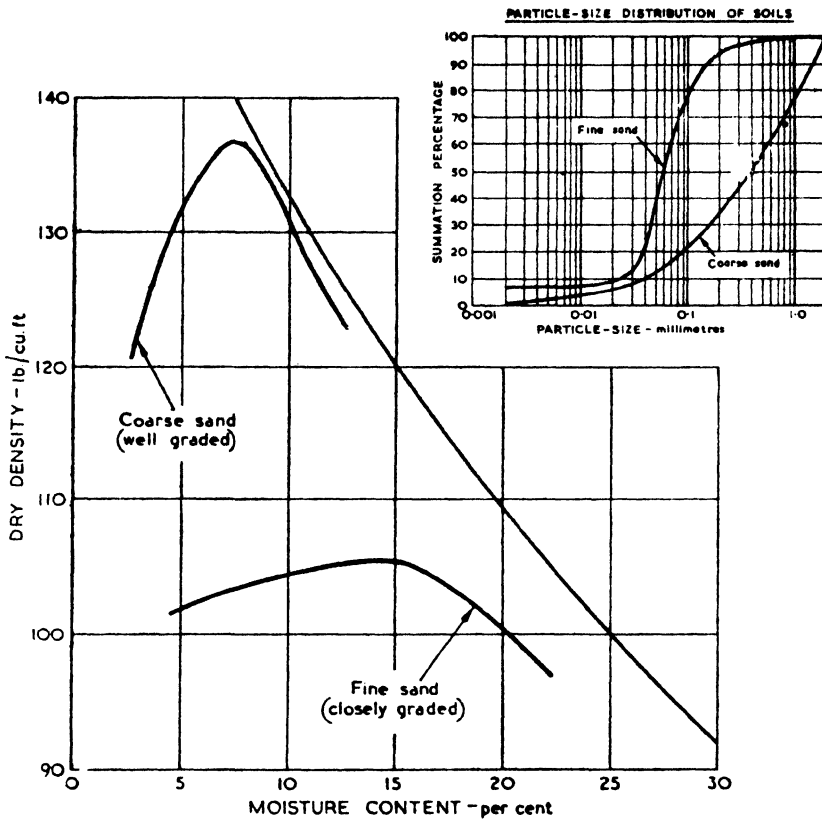



FIG. 9·5 DRY DENSITY/MOISTURE CONTENT CURVES FOR TWO SANDS WITH DIFFERENT PARTICLE-SIZE DISTRIBUTIONS

9·14 Considering Fig. 9·6, for unit volume of soil solids, water and air we have:—

$$1 = \frac{W_s}{\gamma_s V} + \frac{W_w}{\gamma_w V} + \frac{V_a}{100}$$

$$= \frac{\gamma_d}{\gamma_s} + \frac{m\gamma_d}{100\gamma_w} + \frac{V_a}{100}$$

where W_s = Weight of soil solids in a volume V
W_w = Weight of water in a volume V
γ_d = Dry density of the soil
m = Moisture content of soil (%)
γ_s = Density of soil particles
γ_w = Density of water
and V_a = Air voids expressed as a percentage of the total volume.

Therefore, for zero air voids $\frac{1}{\gamma_d} = \frac{1}{\gamma_s} + \frac{m}{100\gamma_w}$

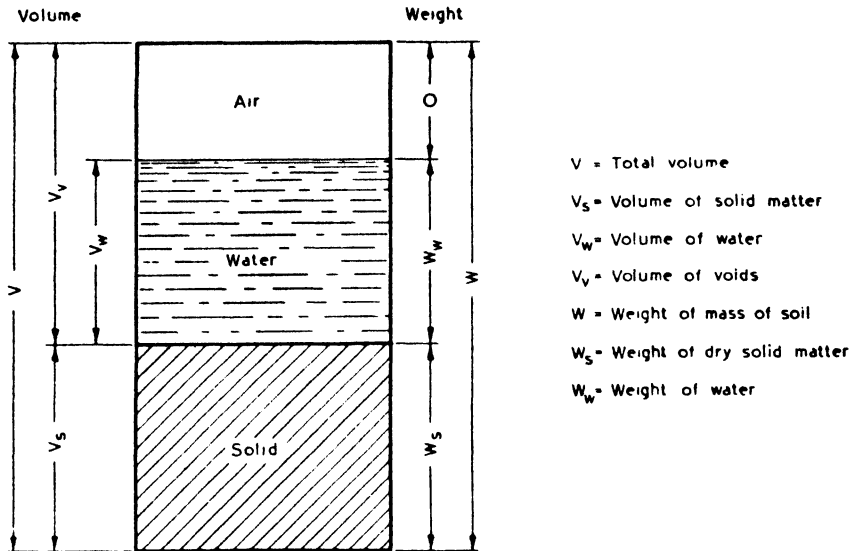



FIG. 9·6 DIAGRAMMATIC REPRESENTATION OF THE COMPOSITION OF SOIL

9·15 If it is required to find lines of constant air voids the relation is

$$\frac{1 - \frac{V_a}{100}}{\gamma_d} = \frac{1}{\gamma_s} + \frac{m}{100\gamma_w}$$

9·16 DETERMINATION OF THE DRY DENSITY/MOISTURE CONTENT RELATIONSHIP FOR SOILS. The dry density of soil is determined after a standard amount of compaction has been applied, and its value is found at several different moisture contents. The compacting energy can be applied either dynamically (by ramming) or statically (by pressure). Four of the most widely used tests are described in detail.

9·17 THE B.S. COMPACTION TEST (PROCTOR TEST, A.A.S.H.O. TEST DESIGNATION T.99-38[3]; B.S. 1377:1948, Test No. 9). This test was developed by R. R. Proctor in 1933 in connexion with the construction of earthfill dams in California. The apparatus used (Plate 9·1A) consists of a cylindrical metal mould having a capacity of $\frac{1}{30}$ cu.ft with an internal diameter of 4 in. and a height of 4·6 in. A detachable collar $2\frac{1}{2}$ in. in height fits on the top of the mould, and the base is also detachable. Soil is compacted in the mould with a metal rammer having a 2-in. diameter circular face and weighing $5\frac{1}{2}$ lb. The rammer is contained in an outer cylindrical sleeve and is designed so that the height of fall of the rammer on to the soil is 12 in. The soil to be used in the test is first air-dried * and passed through a $\frac{3}{4}$-in. B.S. sieve ($\frac{3}{16}$-in. in A.A.S.H.O. test). It is then mixed thoroughly with a small amount of water and compacted into the mould in three equal layers to give a total compacted depth of

*The soil can be dried in an oven if experience shows that this does not materially affect the results.

5 in., each layer being compacted by 25 blows of the rammer dropped through a height of 12 in. The soil is trimmed to the top of the mould and weighed to determine its wet bulk density. A moisture content determination is made on a sample of the soil and the dry density is then calculated. This procedure is repeated at several increasing moisture contents and a compaction curve is obtained as shown in the lower curve in Fig. 9·7.

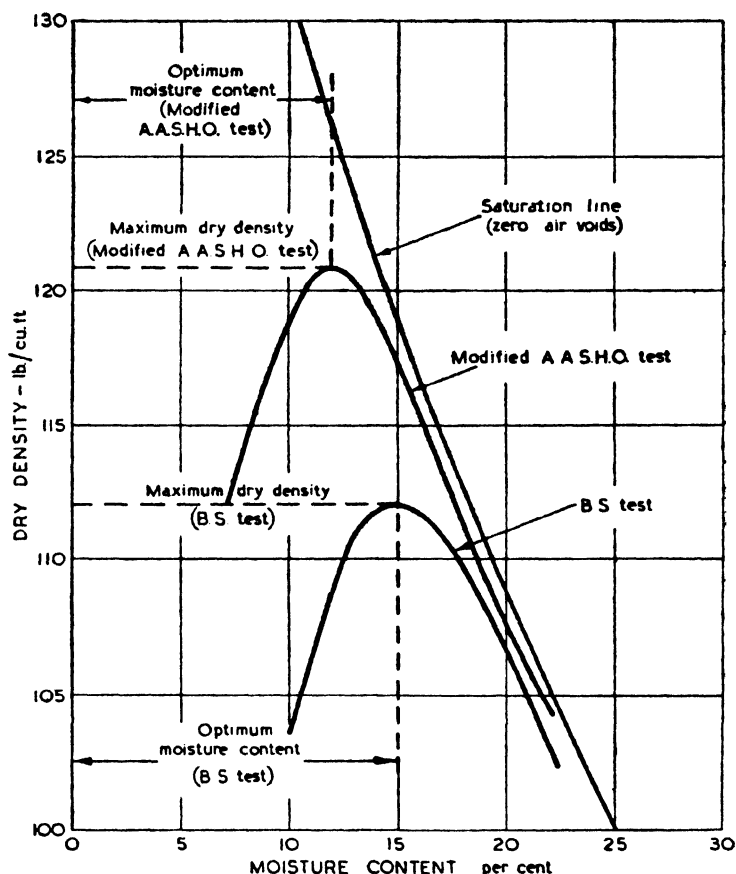



FIG. 9·7 EFFECT OF COMPACTIVE EFFORT ON DRY DENSITY/MOISTURE CONTENT RELATIONSHIP OF A SANDY CLAY SOIL

9·18 The test can be carried out with a mechanical ramming apparatus such as the one designed at the Road Research Laboratory and shown in Plate 9·1B. This apparatus gives a more uniform distribution of the compaction over each layer and results in a smaller scatter of the experimental points on the dry density/moisture content graph.

9·19 THE MODIFIED A.A.S.H.O. COMPACTION TEST[4]. This test was developed to give a heavier standard of compaction for airfield construction. The apparatus used is fundamentally the same as that used for the B.S. compaction test, except that the weight of the rammer is increased to 10 lb. and the cylindrical sleeve

is designed so that the height of fall of the rammer on to the soil is 18 in. The test procedure differs from the standard compaction test in that the soil is compacted into the mould in five equal layers to give a total compacted depth of 5 in., each layer being compacted by 25 blows of the 10-lb. rammer dropped through 18 in. The dry density/moisture content relation is obtained as before, and is shown in the upper curve of Fig. 9·7. The increase in the amount of compaction applied has the effect of increasing the value of the maximum dry density and lowering the optimum moisture content.

9·20 THE CALIFORNIA STATIC LOAD COMPACTION TEST[5]. This test was developed about 1935 by O. J. Porter of the California Division of Highways in connexion with the California bearing ratio test.

9·21 The apparatus used (Plate 9·2) consists of a mould 6 in. in diameter by 8 in. high, fitted with a detachable base plate and a piston 5 in. long. Samples of about 4,000 gm of air-dried* soil are made up from the material passing a $\frac{3}{4}$-in. B.S. sieve. Ift here is material over $\frac{3}{4}$ in. it is replaced by material passing a $\frac{3}{4}$-in. B.S. sieve and retained on a $\frac{3}{16}$-in. B.S. sieve. The samples are thoroughly mixed with sufficient quantities of water so that their moisture contents increase over a convenient range by successive increments of 1 or 2 per cent. Each of these samples is then compacted in the mould in an hydraulic press under a load of 2,000 lb./sq. in. In increasing the pressure from 1,000 to 2,000 lb./sq. in. the rate of strain used is about 0·05 in./min. and the pressure of 2,000 lb./sq. in. is maintained for 1 min. and released over a period of 20 sec. The height of the specimens is measured with a depth gauge and their volume and the wet bulk density are calculated. The moisture content of the specimens is determined and the dry density plotted against the moisture content.

9·22 THE DIETERT TEST[6]. The apparatus used in this test (Plate 9·3A) consists of a mould approximately 2 in. in diameter supported on a metal base bv two pegs. The soil is compacted in the cylinder by means of a piston through which the compacting energy is indirectly applied by dropping, by means of a cam, a cylindrical weight of 18 lb. through a height of 2 in. on to a steel plate rigidly attached to the piston through a rod.

9·23 Several 150-gm samples of air-dried* soil passing the No. 7 B.S. sieve are placed on a glass plate and each sample is brought to a different moisture content by thoroughly mixing with the required amount of water. The moisture contents differ from each other by about 1 per cent. The wet soil is transferred to the cylinder and compacted by the application of 10 blows of the weight, after which the mould is inverted and a further 10 blows are applied.

9·24 Since the diameter of the compacted soil is the same as the known internal diameter of the mould it is necessary to determine only the weight and length of the soil cylinder in order to obtain the bulk density of the soil. The length of the soil cylinder may be determined by ejecting it from the mould and measuring it directly or by finding the difference between the length of the mould and soil cylinder by inserting a metal cylinder having a specially calibrated spiral curve engraved on its cylindrical face. The moisture content

*The soil can be dried in an oven if experience shows that this does not materially affect the results.

of the soil is found by drying the whole cylinder. The same procedure is followed with the other soil samples, and the dry density plotted against moisture content as in the other tests.

Laboratory Investigations relating to Compaction Tests

9·25 THE EFFECT OF STONE CONTENT. Using the A.A.S.H.O. compaction test the effect of the presence of stones upon the compaction of cohesive soil has been investigated by Maddison[7] and other workers at the Road Research Laboratory. Maddison found that the admixture of single-sized aggregate up to about 25 per cent of any one size (1 in.-¾ in., ¾ in.-½ in., ½ in.-⅜ in.) had very little effect on the compaction of the soil mortar, the stone merely acting as a displacer. At higher stone contents the dry density of the soil mortar decreased fairly rapidly and at a stone content of 65 per cent had fallen to a value of 75 per cent of the dry density of the soil when compacted alone. The dry density of the soil/stone mixture as a whole increased with increasing amounts of stone up to a stone content of 40 per cent; at stone contents higher than about 70 per cent the stones were in contact with one another, and so prevented any compaction of the soil mortar. The optimum moisture content of the soil mortar increased as the dry density decreased with addition of stone.

9·26 In later work an aggregate graded between ¾ in. and No. 7 B.S. sieve was used. The results obtained were similar to Maddison's, but the dry density of the soil mortar decreased on the addition of even small proportions of aggregate, although this decrease was not considerable until more than 45

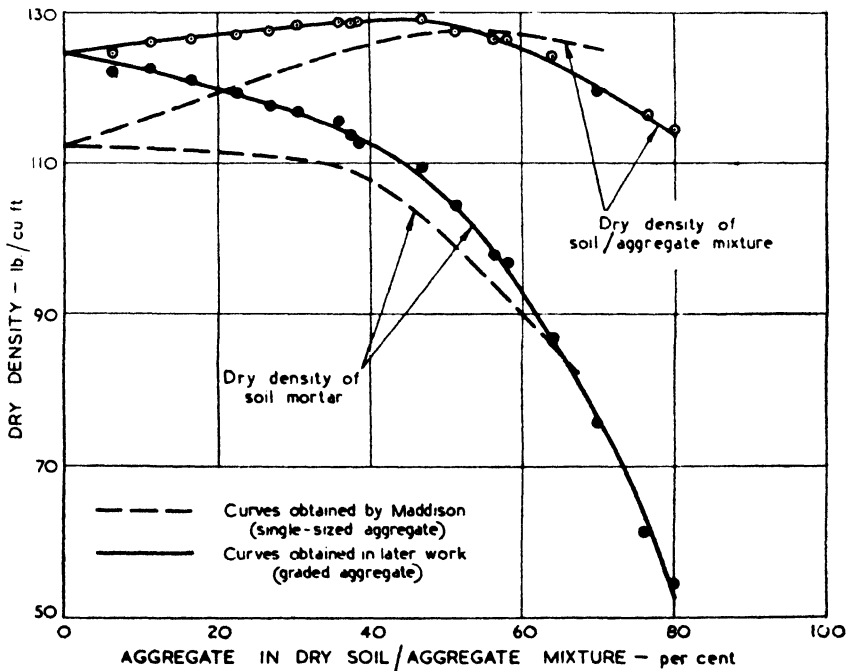



FIG. 9·8 COMPACTION OF SOIL MORTAR AT OPTIMUM MOISTURE CONTENT WITH DIFFERENT PERCENTAGES OF AGGREGATE

per cent of aggregate had been added. Also the dry density of the soil/aggregate mixture increased only slightly with increasing percentages of aggregate up to 50 per cent and for higher percentages decreased (Fig. 9·8).

9·27 In the A.A.S.H.O. compaction test it is assumed that material retained on the $\frac{3}{16}$-in. B.S. sieve acts as a displacer, and the maximum dry density and optimum moisture content of stony soils are adjusted accordingly. The above results show that this assumption is generally inadmissible and that it is desirable to use the entire soil.

9·28 Further investigations of the A.A.S.H.O. compaction test were therefore made using various sieved-out fractions from a gravel-sand-clay mixture all passing the $1\frac{1}{2}$-in. B.S. sieve. The results showed that the reproducibility of the test was not affected by increasing the maximum size of the material to $\frac{3}{4}$-in. The presence of the coarse material hindered the full compaction of the finer material, the values of the maximum dry density being practically unchanged by the presence of the coarser material.

9·29 As a result of this work, the B.S. compaction test was devised to be similar to the A.A.S.H.O. test but for material passing the $\frac{3}{4}$-in. B.S. sieve.

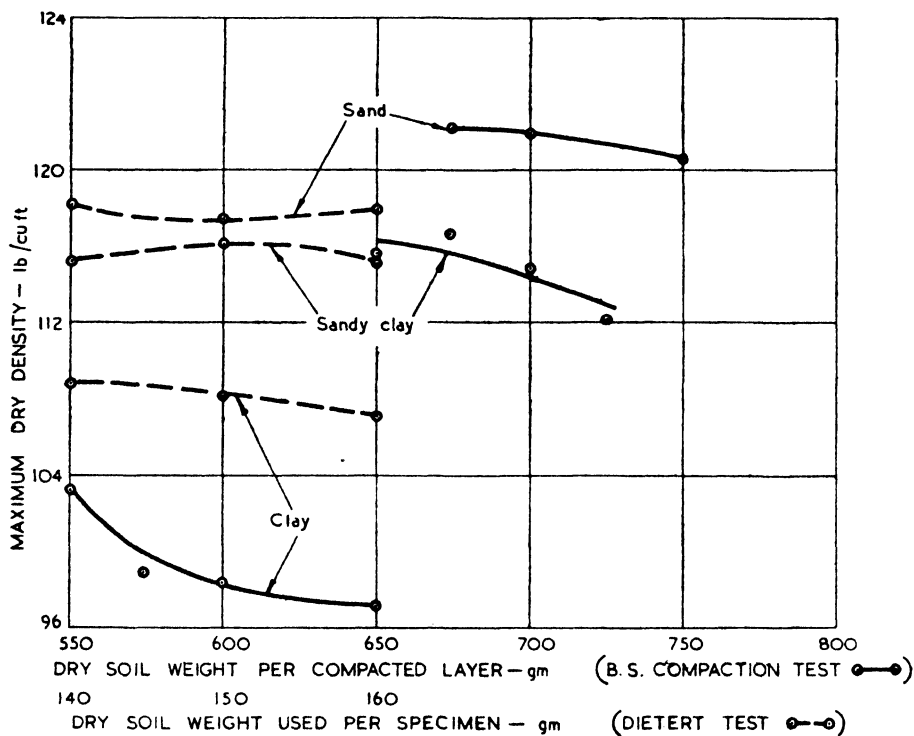



FIG. 9·9 VARIATION OF MAXIMUM DRY DENSITY WITH DRY WEIGHT OF COMPACTED SOIL IN LABORATORY COMPACTION TESTS

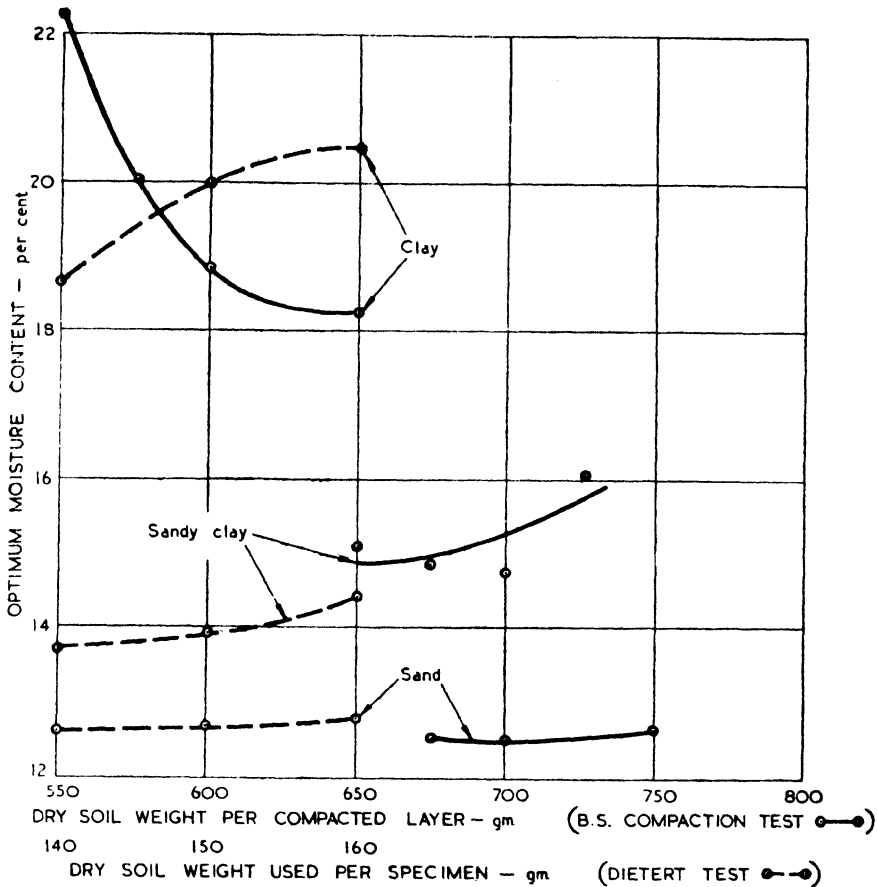



FIG 9·10 VARIATION OF OPTIMUM MOISTURE CONTENT WITH DRY WEIGHT OF COMPACTED SOIL IN LABORATORY COMPACTION TESTS

9·30 COMPARISON OF THE B.S. COMPACTION TEST AND DIETERT TEST. An investigation was carried out at the Road Research Laboratory to compare the results obtained in the B.S. compaction test and the Dietert test for three soils ranging from a sand to a heavy clay. Some of the results of this investigation are illustrated in Figs. 9·9 and 9·10 and the properties of the soils used are given in Table 9·1. Both tests were found to agree fairly well with regard to optimum moisture content, but the maximum dry densities differed considerably for soils having a high sand or high clay content. Variations also occurred in the optimum moisture content and maximum dry density when the dry weight of soil compacted was varied in either test. The investigation also showed that the values of the optimum moisture content and maximum dry density obtained with the B.S. compaction test, when quoted to the nearest whole number, do not differ on repetition of the test by more than ± 1 per cent and ± 1 lb./cu.ft respectively, when the test is carefully done.

TABLE 9·1

PROPERTIES OF SOILS USED IN COMPARISON OF B.S. AND DIETERT COMPACTION TESTS

Soil type	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Particle-size distribution %		
				Sand (2.0-0.06 mm.)	Silt (0.06-0.002 mm.)	Clay (<0.002 mm.)
Sand	Non-plastic			77	16	7
Sandy clay	27	19	8	30	48	22
Clay	53	23	30	10	33	57

9·31 COMPARISON OF THE RESULTS OBTAINED FROM THE B.S. COMPACTION TEST AND THE MODIFIED A.A.S.H.O. test. As previously mentioned, the effect of the increased amount of compaction applied in the modified A.A.S.H.O. test (25 blows of a 10-lb. rammer falling 18 in. on each of five layers) compared with the compaction applied in the B.S. test (25 blows of a 5·5-lb. rammer falling 12 in. on each of three layers) is to increase the maximum dry density and reduce the optimum moisture content. In Table 9·2 the results obtained with these tests are summarized for five soils ranging from a well graded gravel-sand-clay mixture to a heavy clay. The increase in the amount of compaction had the greatest effect on heavy and silty clays, resulting in a large increase in maximum dry density and a correspondingly large decrease in optimum moisture content.

TABLE 9·2

COMPARISON OF RESULTS OF THE B.S. AND MODIFIED A.A.S.H.O. COMPACTION TESTS

Type of soil	Average results of B.S. compaction test		Average effect of modified A.A.S.H.O. test on:—	
	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)
Heavy clay	97	28	Increased by 20	Decreased by 10
Silty clay	104	21	Increased by 17	Decreased by 9
Sandy clay	115	14	Increased by 13	Decreased by 3
Sand	121	11	Increased by 9	Decreased by 2
Gravel-sand-clay ...	129	9	Increased by 8	Decreased by 1

9·32 COMPARISON OF METHODS OF CARRYING OUT THE MODIFIED A.A.S.H.O. TEST. This investigation was carried out to determine whether the maximum dry density and optimum moisture content obtained with the modified A.A.S.H.O. test could be obtained if the procedure were altered by using a heavier rammer falling through only 12 in. Two weights of rammer were tried: a 15-lb. rammer giving the same amount of applied energy as in the normal test and a 12½-lb. rammer giving the same amount of momentum.

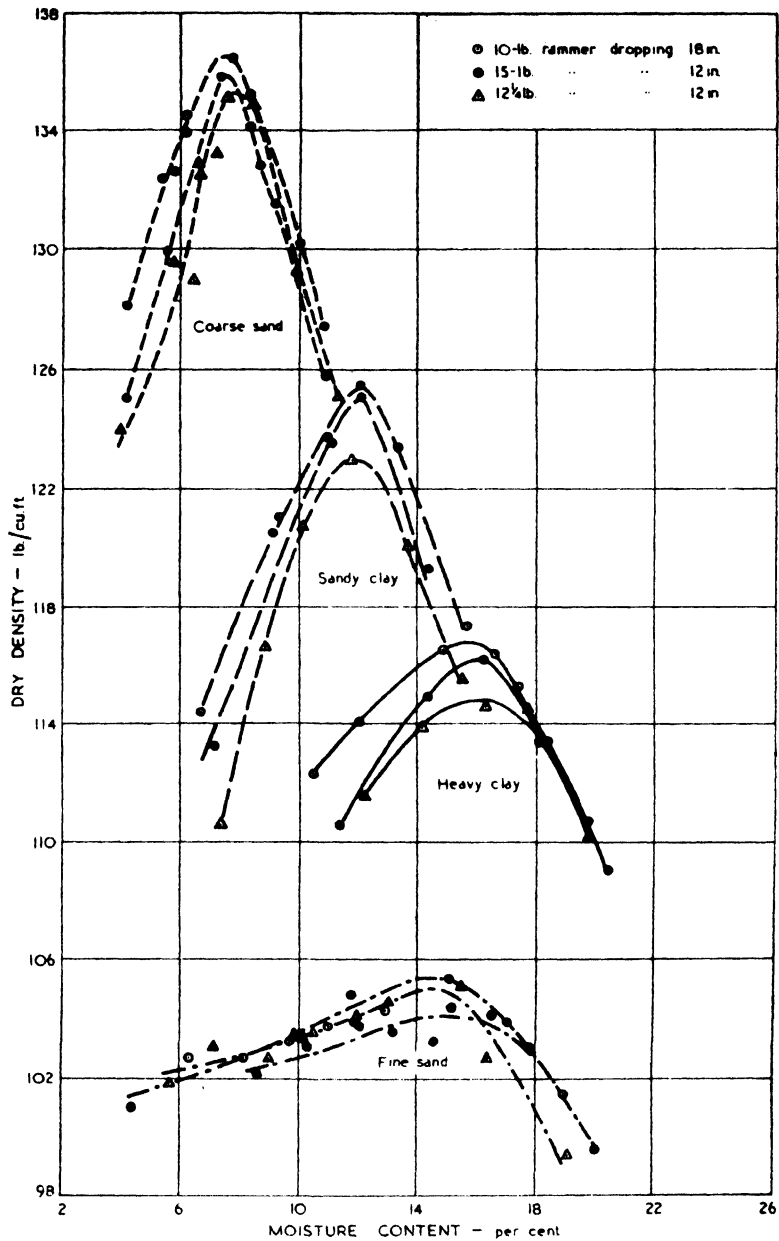



FIG. 9·11 DRY DENSITY/MOISTURE CONTENT RELATIONSHIPS FOR FOUR SOIL TYPES USING RAMMERS OF DIFFERENT WEIGHTS DROPPING THROUGH DIFFERENT HEIGHTS

The dry density/moisture content curves obtained for the three methods of compaction on four soils ranging from a course sand to a heavy clay are shown in Fig. 9·11.

9·33 When the energy was kept constant, it was found that the maximum dry densities obtained by the two procedures differed by less than 1 lb./cu.ft and the optimum moisture contents by less than ½ per cent. When the momentum was kept constant the difference, although slightly greater, was still within the limits of experimental error that would be expected between repeat results.

9·34 It is concluded that the use of a 15-lb. rammer dropped through 12 in. is a satisfactory alternative procedure to the modified A.A.S.H.O. test. This facilitates the design of a mechanical apparatus for ramming the soil for both the B.S. and modified A.A.S.H.O. compaction tests.

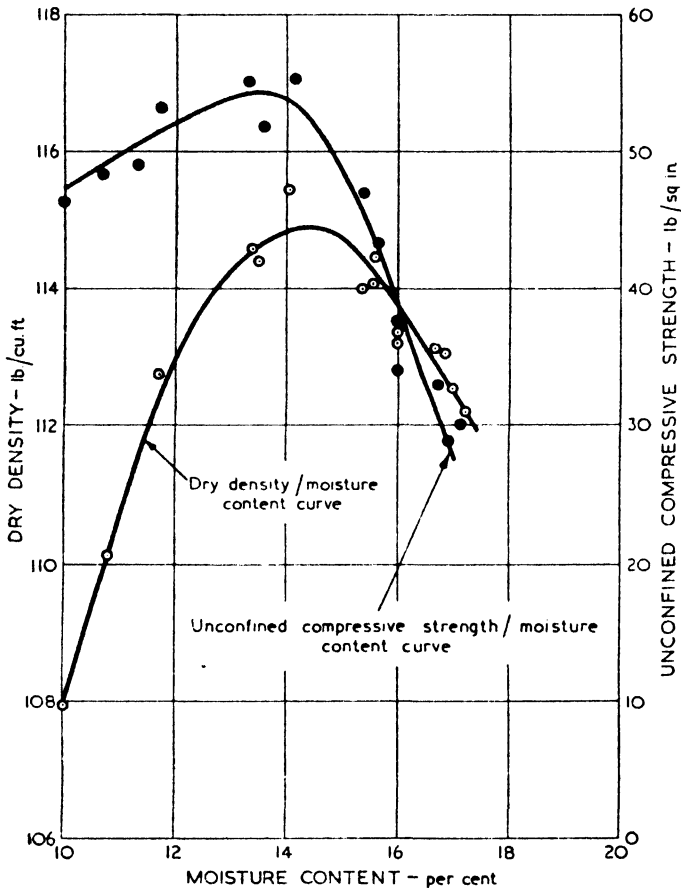



FIG. 9·12 COMPARISON OF UNCONFINED COMPRESSIVE STRENGTH/MOISTURE CONTENT CURVE AND DRY DENSITY/ MOISTURE CONTENT CURVE FOR A SANDY CLAY SOIL USING B.S. COMPACTION PROCEDURE

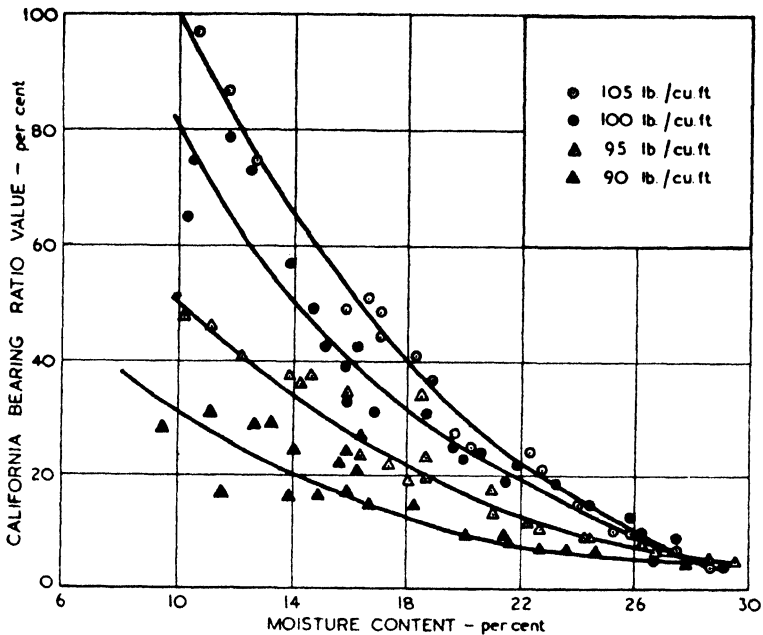



FIG. 9·13 RELATIONSHIP BETWEEN THE CALIFORNIA BEARING RATIO OF A HEAVY CLAY AND ITS DRY DENSITY AND MOISTURE CONTENT

Effect of Compaction on Properties of Soil

9·35 The object in compacting a soil is to improve its properties, and in particular, to increase its strength and bearing capacity, reduce its compressibility and decrease its ability to absorb water. The effect of compaction on these properties of soil is illustrated in the following paragraphs by data obtained at the Road Research Laboratory.

9·36 UNCONFINED COMPRESSIVE STRENGTH (see Chapter 19). Curves relating the unconfined compressive strength of a sandy clay soil with its moisture content and dry density are shown in Fig. 19·18 and Fig. 9·12. Both figures illustrate the importance of compacting the soil properly to obtain a high soil strength. Fig. 9·12 shows that for specimens made up at moisture contents and dry densities corresponding to points on the B.S. compaction curve, the maximum unconfined compressive strength occurred slightly below the optimum moisture content.

9·37 CALIFORNIA BEARING RATIO (see Chapter 19). Typical curves for the relationship between California bearing ratio, moisture content and dry density are shown in Fig. 9·13 and 9·14. These curves are very similar to those obtained for unconfined compressive strength, and it would appear that in order to obtain maximum strength it is desirable to compact a soil at a moisture content less than the optimum. However, owing to the increased percentage of air voids present in such a case, the soil is in a condition which is more susceptible to the absorption of moisture, so its final state might be considerably worse than it would have been had it been compacted at the optimum moisture content.

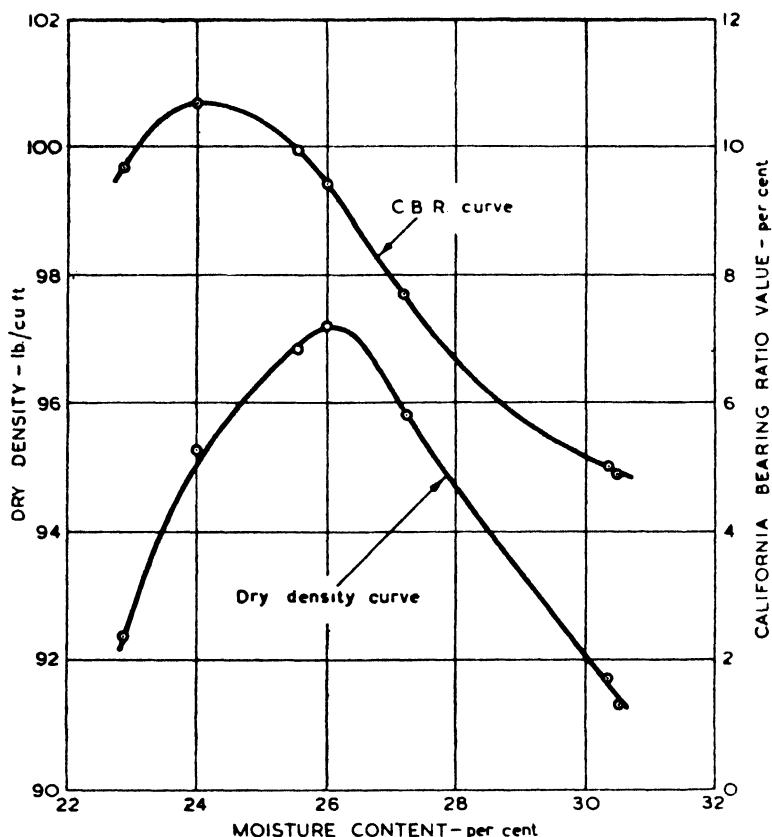



FIG. 9·14 DRY DENSITY/MOISTURE CONTENT AND C.B.R./MOISTURE CONTENT RELATIONSHIPS FOR A HEAVY CLAY USING THE B.S.COMPACTION PROCEDURE

9·38 WATER ABSORPTION (see Chapter 2). The absorption of water by cohesive soils causes swelling and conversely drying causes shrinkage. These volume changes have been studied for cohesive soils, specimens of which were made up to correspond to different points on the dry density/moisture content curves for B.S. compaction. The specimens were made up in pairs, one being dried and the other being wetted. Typical results are given in Fig. 9·15. It will be seen that the drying and wetting curves were continuous and became steeper as the air voids decreased. The maximum changes in volume occurred when the specimens were made up in a saturated condition, but the rate of absorption of water decreased as the percentage of air voids decreased. Although smaller volume changes occurred in soils with large air contents, after saturation they had lower densities and higher moisture contents than those with small air contents.

9·39 It is therefore considered inadvisable to compact cohesive soil subgrades below their optimum moisture content in cases where they are likely to be subject to the ingress of moisture during the life of the road.

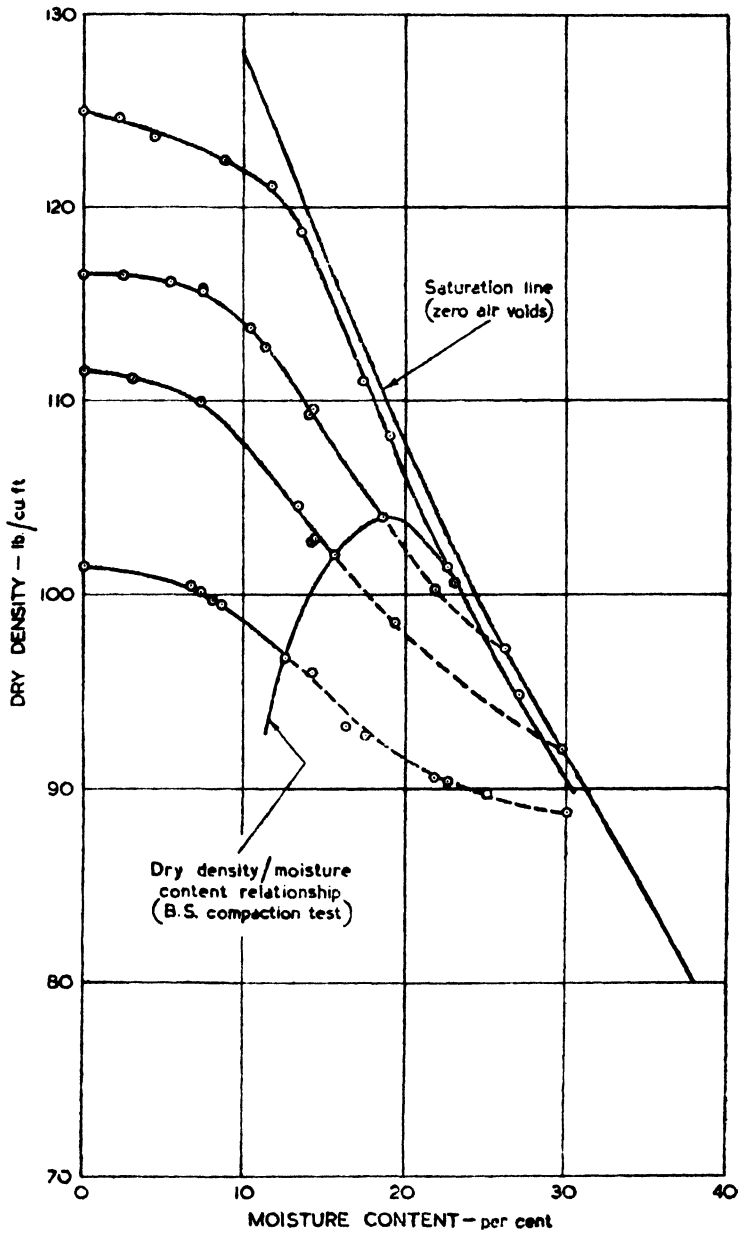



FIG. 9·15 RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT OF A SANDY CLAY SOIL WHEN ALLOWED SLOWLY TO BECOME DRIER OR WETTER

MEASUREMENT OF COMPACTION IN THE FIELD

Measurement of Dry Density

9·40 The usual method of measuring compaction in the field is to determine the dry density of the soil *in situ*. There are four main methods of making this determination, and the procedure involved in all cases is to determine the weight and moisture content of soil removed from an approximately cylindrical cavity whose volume is then measured. These methods are now described in detail.

9·41 CORE-CUTTER METHOD (B.S. 1377:1948, TEST NO. 10C). The apparatus required is shown diagrammatically in Fig. 9·16. The cutter is rammed into the soil, the dolly being placed over the cutter during ramming to prevent burring the edges of the cutter, until the dolly is just proud of the surface. The cutter containing the soil is then dug out of the ground and any soil extruding from its ends is trimmed off so that the cutter contains a volume of soil equal to its internal volume which is determined from the dimensions of the cutter. The weight of the contained soil is found and its moisture content determined. From the weight, volume and moisture content of the soil sample the dry density is readily calculated.

9·42 SAND-REPLACEMENT METHOD (B.S. 1377:1948, TEST NO. 10A). A drawing of the pouring cylinder used in this method is shown in Fig. 9·17 and a photograph of the apparatus in Plate 9·3B. The cylinder contains dry sand graded between the No. 25 and No. 52 B.S. sieves. A hole about 4 in. in diameter is excavated with suitable tools to the depth of layer being tested, and the weight of soil removed is determined and a moisture content sample taken. Sand is run into the hole from the cylinder and the weight of sand in the hole determined from the difference in weight of the cylinder before and after filling the hole, allowing for the sand contained in the cone below the valve. The volume of the sand required to fill the hole is calculated from the known weight and bulk density of the sand, which is calibrated by filling a can of known volume (Plate 9·3B) under similar conditions.

9·43 VOLUMENOMETER METHOD (B.S. 1377:1948, TEST NO. 10D). The apparatus required is shown in Plate 9·4A. A lump of soil approximately 18 cu. in. in volume is cut from the ground, taking care not to compress the soil. The lump is trimmed clear of loose material, weighed, coated with a thin layer of paraffin wax and reweighed. The volume of the waxed soil specimen is found from the volume of water displaced when it is immersed in the volumenometer, the syphon arrangement ensuring a constant water level. The actual volume of the soil specimen is the volume of water displaced less the volume of the paraffin wax. A sample of soil is cut from the specimen and its moisture content determined.

9·44 RUBBER-BALLOON METHOD[8]. The apparatus required is shown in Plate 9·4B. A cylindrical hole about 4 in. in diameter is excavated with suitable tools to the depth of the layer being tested, and the soil removed is weighed and its moisture content determined.

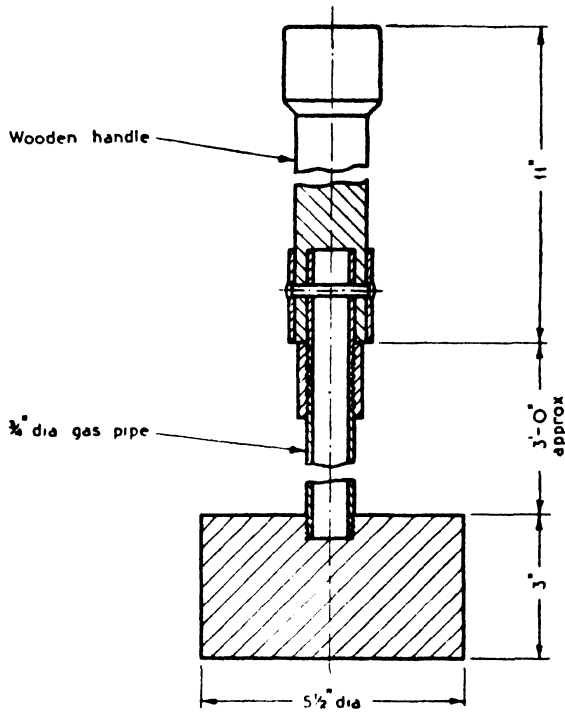
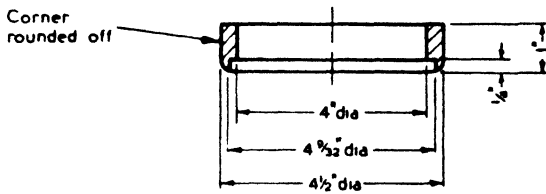
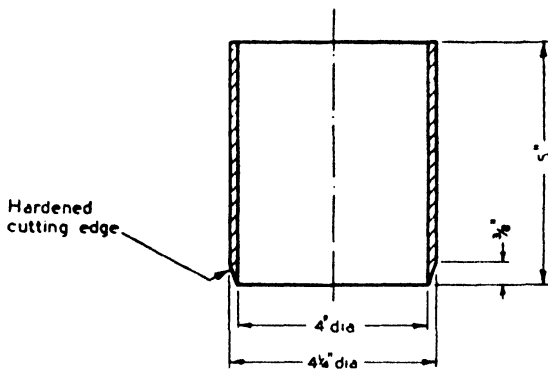



FIG. 9·16 CORE-CUTTER APPARATUS
Used for the determination of dry density of soil in the field

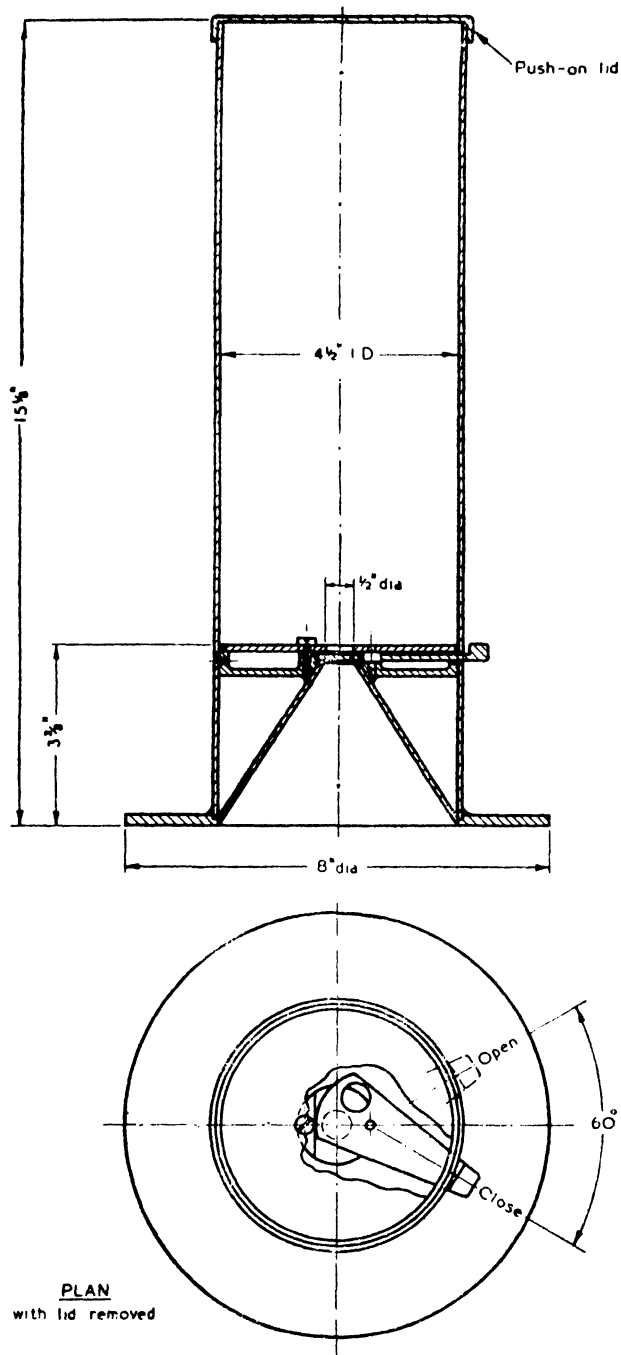



FIG. 9·17 SAND-POURING CYLINDER
Used in the determination of dry density of soil in the field

9·45 To determine the volume of the hole, the apparatus is placed over the hole and water is forced into the balloon situated in the hole, until the base of the instrument is raised ¾ in. off the ground. The air-cock is then closed and both feet are placed firmly on the base-plate so that the balloon is forced into any irregularities in the hole. The volume of the hole is found from the difference between the initial and final water levels in the glass cylinder.

Variation of Density of Compacted Soil

9·46 The measured dry density of a soil will vary slightly from place to place, even over a comparatively small area. This variation may be due partly to actual variations in soil density brought about by non-uniform compaction or by minor changes in soil type or moisture content, and partly to errors inherent in the method of density measurement used and lack of skill on the part of the operator.

9·47 If compaction is to be controlled by density measurements, the control should not be based on the result of any one test. A number of tests should be made and the results should be analysed by a statistical method[9] to determine the standard deviation and the limits within which the true mean will lie for a probability of 9 chances in 10 of the actual mean.

9·48 It is considered that 10 density determinations should be used as a basis for analysis, one measurement being made for each 1,000 sq. yd. The number of determinations to any area will of course depend on the nature of the work, and the degree of accuracy of the results required.

9·49 For most classes of work standard deviations of 5 lb./cu.ft for fine grained soils and 10 lb./cu.ft for coarse-grained soils are permissible, and the mean dry density should be equal to, or exceed, the specified density.

Investigations of Factors affecting the Results of the Measurement of Field Densities

Core-cutter Method

9·50 Effect of wall thickness of core-cutter. A comparison was made at two separate sites of the densities of the soil measured by core-cutters having the same internal dimensions (4 in. diameter x 5 in. high) but wall thicknesses of $\frac{1}{16}$, $\frac{1}{8}$, $\frac{3}{16}$ and $\frac{1}{4}$ in.; the results are given in Table 9·3.

TABLE 9·3

EFFECT OF WALL THICKNESS OF CORE-CUTTER

Wall thickness (in.)	Dry density (lb./cu.ft)	
	Site 1	Site 2
$\frac{1}{4}$	102.7	98.1
$\frac{3}{16}$	103.0	98.0
$\frac{1}{8}$	103.2	99.8
$\frac{1}{16}$	105.2	100.3

9·51 These results suggest that the compression of the soil caused by the insertion of the core-cutter into the ground is outweighed by an expansion of the soil due to stresses set up as the core is driven into the ground. To obtain an accurate measurement of dry density with the core-cutter method it is clear that a cutter with a thin wall should be used.

9·52 EFFECT OF DIAMETER OF CORE-CUTTER. No appreciable difference in density or in the standard deviation of repeat results was found when comparing core-cutters of 4-in. and 6-in. internal diameter, having a wall thickness of ⅛ in. These results indicated that a better average value of dry density cannot be obtained by using the 6-in. instead of the 4-in. diameter core-cutter.

Sand-replacement Method

9·53 EFFECT OF DEPTH OF CALIBRATING CAN ON THE BULK DENSITY OF SAND. The bulk density of the sand was found to decrease by about 1 per cent for a 1-in. decrease in the depth of the can. The depth of the calibrating can used should therefore be approximately equal to the depth of the hole used in the test.

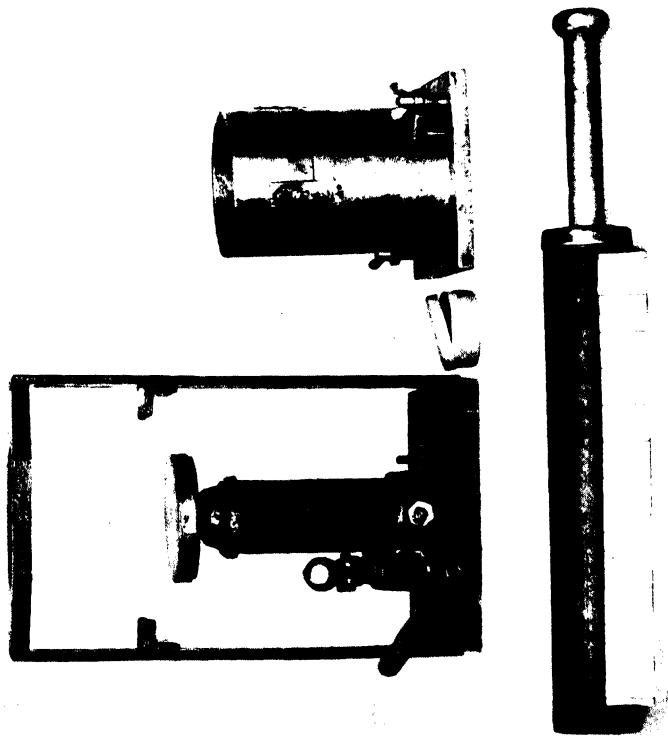
9·54 EFFECT OF VARIATIONS IN THE INITIAL LEVEL OF THE SAND IN THE CYLINDER ON THE BULK DENSITY OF THE SAND. The bulk density of the sand was found to decrease by about 1 per cent for a 2-in. decrease in the level of the sand in the cylinder. The height of the sand in the cylinder should therefore be kept approximately the same at the beginning of each test, including the calibration test.

9·55 EFFECT OF THE PARTICLE-SIZE DISTRIBUTION OF SAND ON THE REPRODUCIBILITY OF TESTS. It was found important that the sand should be dry and clean, as otherwise its bulk density varied considerably. Sand of size distribution between the No. 25 and No. 52 B.S. sieves gave the most reproducible results, and variations in dry density obtained with this size distribution were accounted for entirely by the limits of accuracy of weighing (± 1 gm).

9·56 The reproducibility obtained with other size distributions tested was very good, and the non-availability of the No. 25 - No. 52 B.S. sieve sand would not seriously detract from the value of the method. The results of this investigation are given in Table 9·4. It is essential to use a closely graded sand in order to prevent segregation occurring.

TABLE 9·4

REPRODUCIBILITY OF CALIBRATION TESTS CARRIED OUT USING SANDS OF DIFFERENT PARTICLE-SIZE DISTRIBUTION

Particle-size distribution of sand between B.S. sieves:-	Maximum variation from mean value (%)	
	Weight in conical funnel (gm)	Weight in calibrating can (gm)
No. 14 - No. 25	0.4	0.3
No. 25 - No. 52	0.2	0.1
No. 52 - No. 100	0.6	0.2
Passing No. 100	0.7	0.2

(a) Standard apparatus

(b) Mechanical ramming apparatus

BRITISH STANDARD COMPACTION TEST

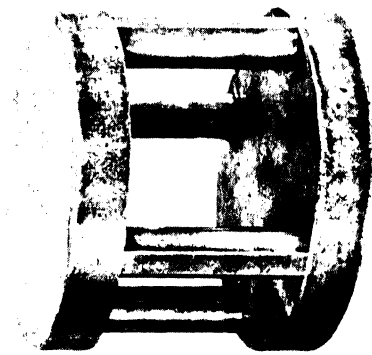
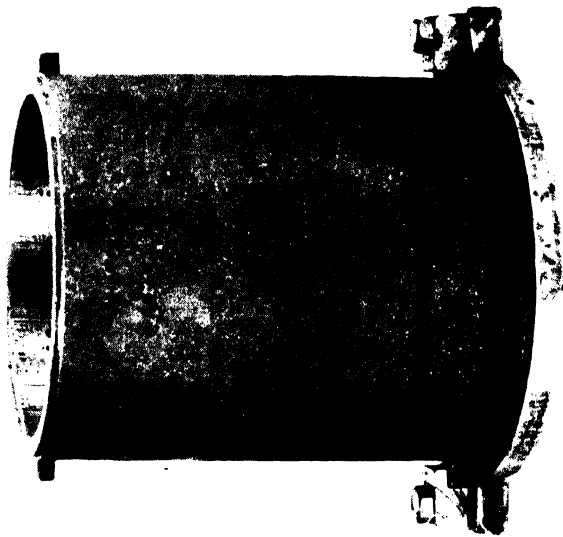
PLATE 9.1

APPARATUS FOR CALIFORNIA STATIC-LOAD COMPACTION TEST

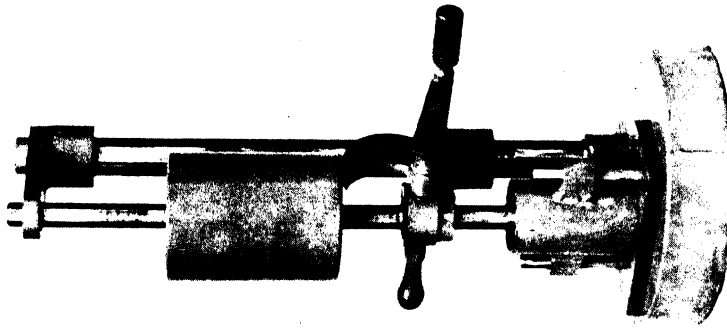
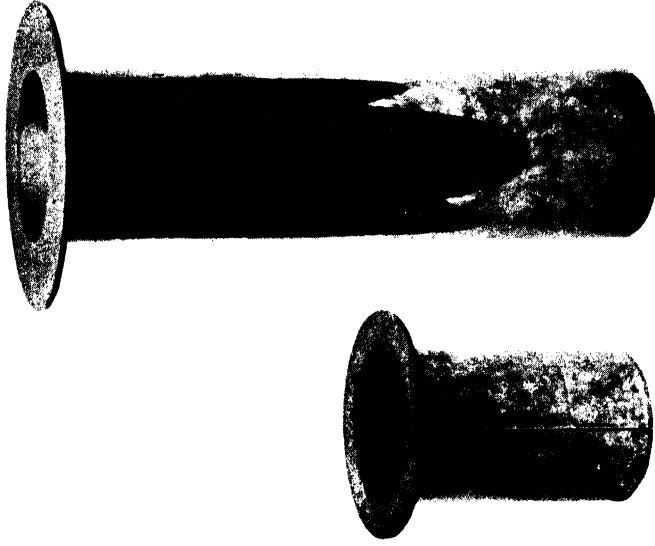
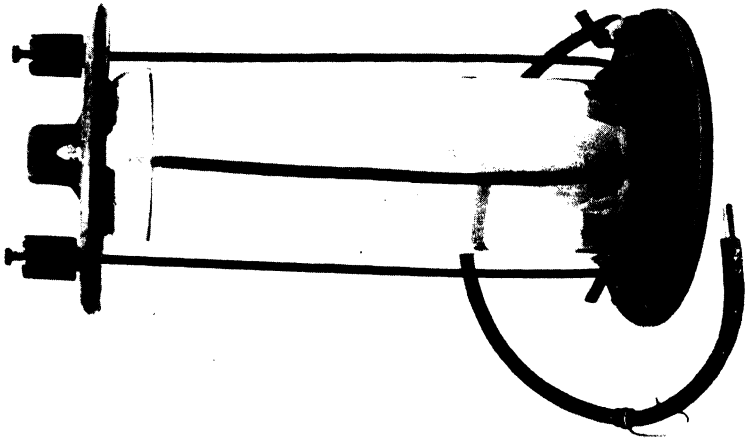
PLATE 9·2

(A) THE DIETERT COMPACTOR

(B) SAND-POURING CYLINDER AND CALIBRATING CAN

PLATE 9·3

(A) VOLUMENOMETER

used in the determination of dry density of soil by water displacement

(B) BALLOON APPARATUS

used in the determination of dry density of soil in the field

PLATE 9·4

(A) SOIL PENETROMETER
(PROCTOR NEEDLE)

(B) MODIFIED SOIL PENETROMETER
(HYDRAULIC TYPE)

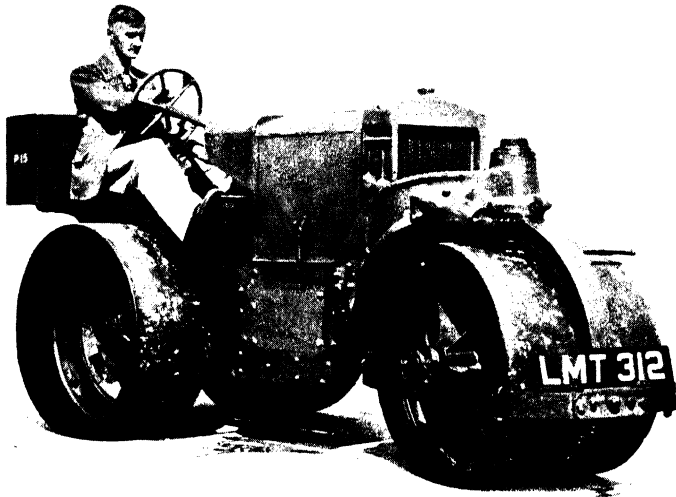



PLATE 9·5

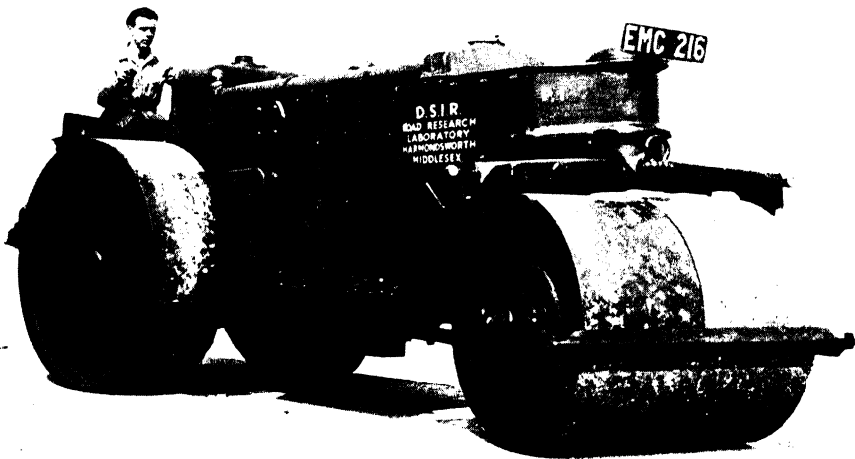
(A) $2\frac{3}{4}$-TON SMOOTH-WHEEL ROLLER

(B) 8-TON SMOOTH-WHEEL ROLLER

PLATE 9·6

PNEUMATIC-TYRED ROLLER

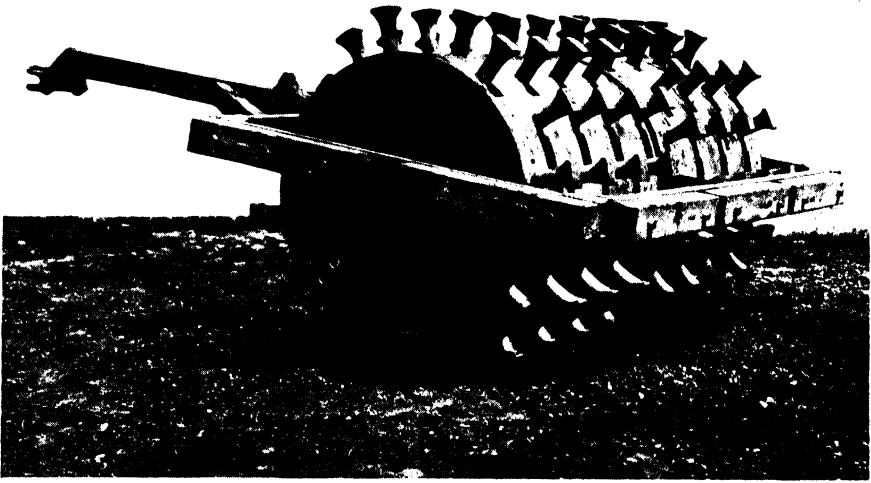
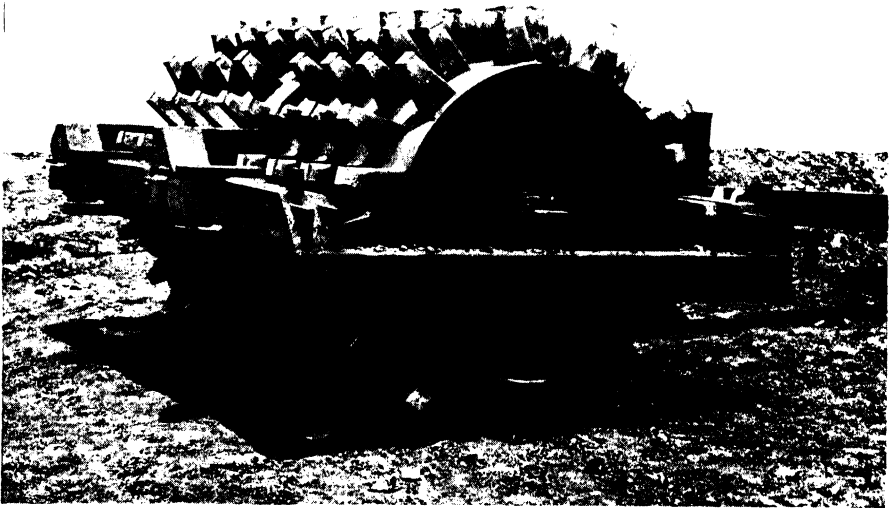
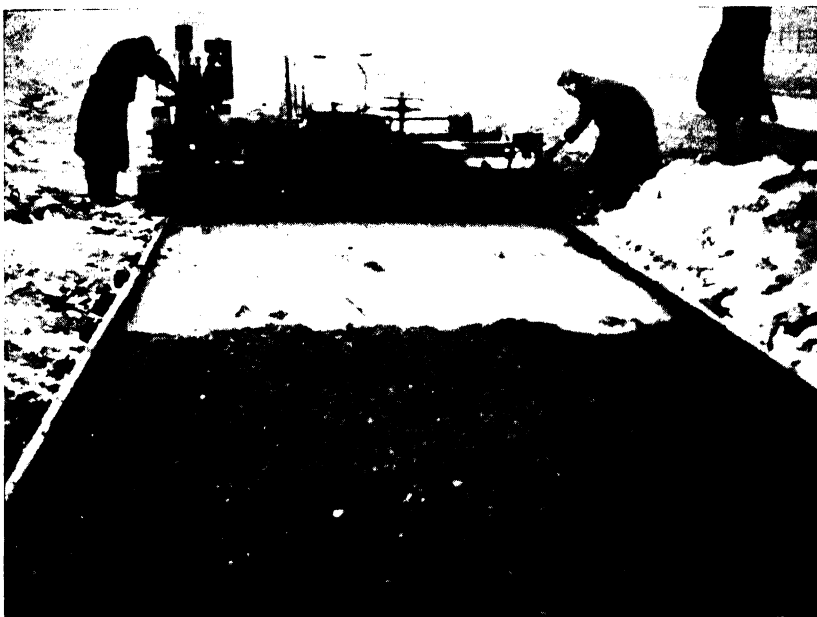
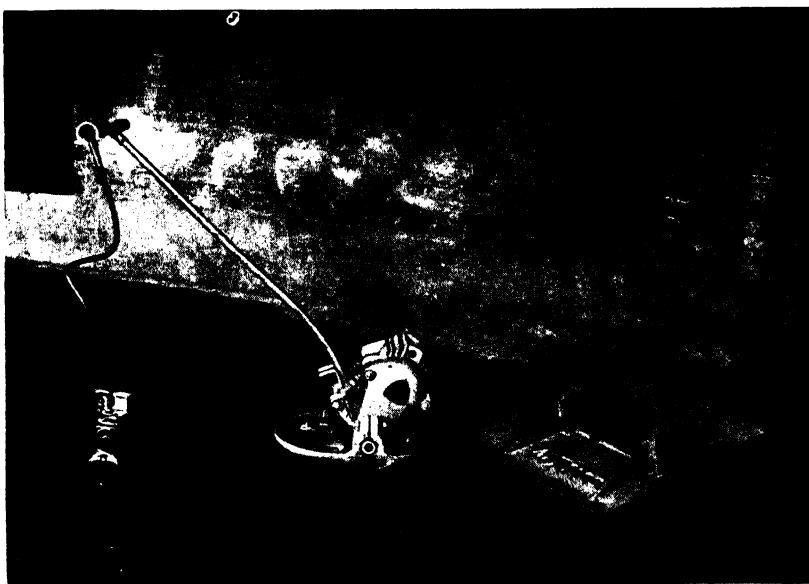



PLATE 9·7

(a) Club-foot type

(b) Taper-foot type

SHEEPSFOOT ROLLERS

PLATE 9·8

(A) TRAVELLING VIBRATORY CONCRETE ROAD FINISHER
(Vibrating screed)

(B) VIBRATING MECHANISM ATTACHED TO SMALL PLATE
Large plate also shown

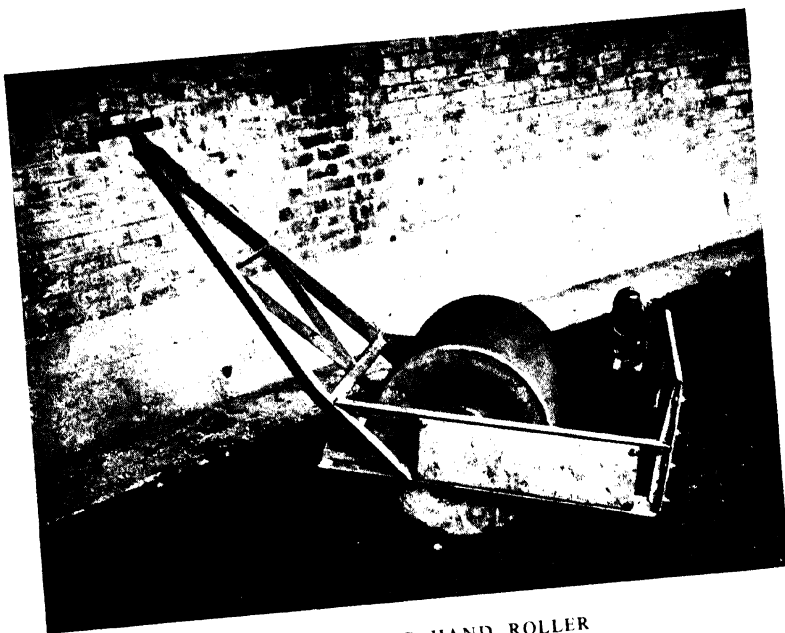



PLATE 9·9

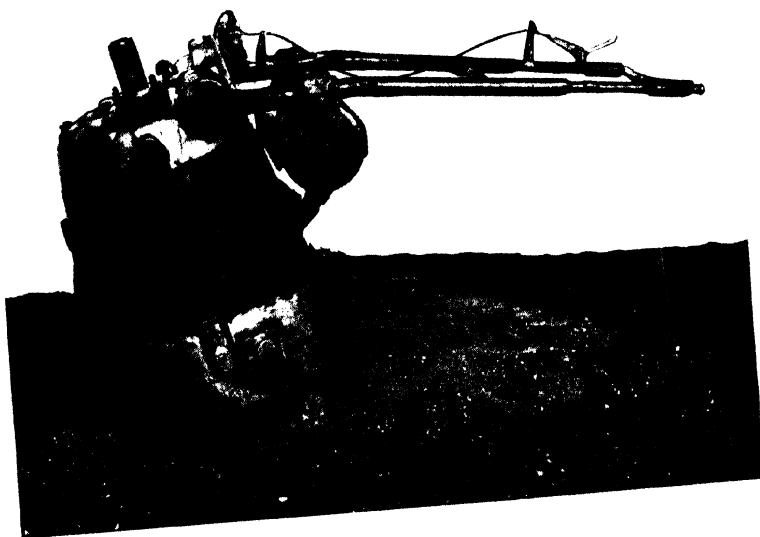
(A) VIBRATING HAND ROLLER

(B) ½-TON FROG-RAMMER

PLATE 9·10

Comparison of the Results obtained from the Four Methods

9·57 When used on a fine-grained cohesive soil, to which all methods are applicable, it was found that the core-cutter (4 in. in diameter and ⅛-in. wall thickness) method and the volumenometer method gave results which were in close agreement, while the results with the sand-replacement method were about 2 per cent less. This difference is probably due to the conical spout of the sand cylinder being slightly proud of the ground. The balloon apparatus gave relatively unreliable results, probably owing to the inaccuracy with which the balloon fits the hole; the fit depends on the air pressure and cannot be readily controlled.

Practical Advantages and Disadvantages of the Methods

9·58 These may be summarized as follows:—

(1) The core-cutter method is convenient and quick. The cutting edge is easily damaged and needs frequent re-sharpening. The method works best on soft, cohesive soils and cannot be used on stony or non-cohesive soils.

(2) The sand-replacement method is relatively slow, but it can be used on any type of soil.

(3) The volumenometer method is a lengthy process and it can only be used on cohesive soils. The samples must not be permitted to dry between the time when they are cut from the ground and when their volume is determined.

(4) The balloon apparatus is convenient and quick, but the results are not very reproducible owing to the difficulty of controlling the air pressure and ensuring that the balloon conforms to the shape of the hole. This difficulty might be overcome by increasing the size of the apparatus so that the error due to the balloon not conforming to the surface irregularities of the hole would be minimized; with a larger balloon apparatus, the method might be applicable to stony soils.

Measurement of Moisture Content by Proctor Needle Method

9·59 The usual methods of measuring the moisture content of soil have been fully described in Chapter 3. In controlling the compaction of earthworks, the rapid determination of the soil moisture content is sometimes made with the Proctor needle (Plate 9·5A). It consists of a needle attached to a spring-loaded plunger, the stem of which is calibrated to read in pounds. The needle is supplied with a series of points so that a wide range of penetration resistances can be measured.

9·60 A modified form of the apparatus operating hydraulically has been designed in order to obtain a greater control over the rate of penetration and this is shown in Plate 9·5B.

9·61 A calibrating curve (penetration resistance/moisture content) for the soil is made during the B.S. compaction test by determining the penetration resistance of the soil when compacted into the mould at each moisture content. This is done by reading the force required to drive a suitably sized needle into the soil at a rate of ½ in./sec. to a depth of 3 in. The penetration resistance is calculated from the force and the area of the needle and is plotted against the moisture content as in Fig. 9·18. To determine the moisture content in

the field, a sample of the wet soil is compacted into the standard mould under the same conditions as in the B.S. compaction test, and the penetration resistance determined. The moisture content is read off from the calibration curve.

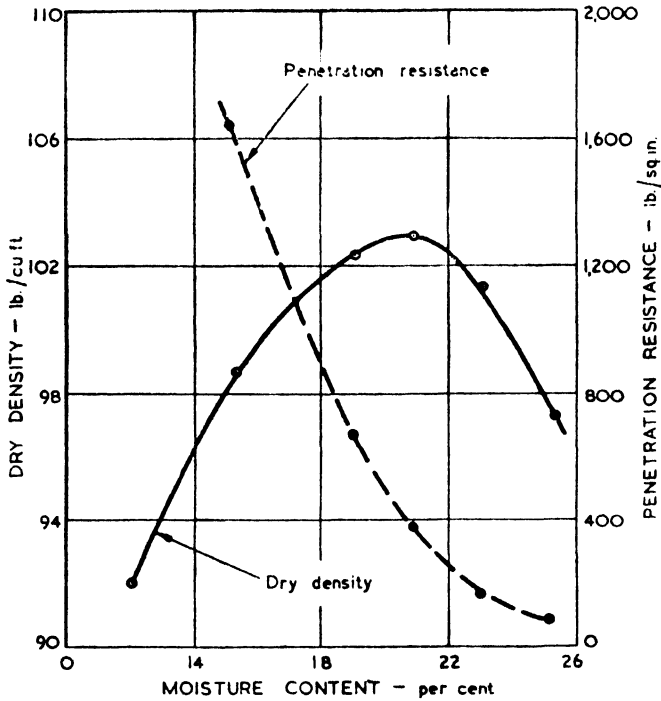



FIG. 9·18 RELATIONSHIP BETWEEN PENETRATION RESISTANCE OF A PROCTOR NEEDLE AND MOISTURE CONTENT OF A CLAY SOIL COMPARED WITH DRY DENSITY/MOISTURE CONTENT CURVE

9·62 This method is very rapid, and on fine-grained cohesive soils it is quite accurate. It is not, however, applicable to soils which have an appreciable amount of material retained on a No. 7 B.S. sieve, or to cohesionless sands, on account of the variable nature of the results.

FIELD COMPACTION

9·63 In the field, soil is compacted by applying energy in one of three ways, which, in order of duration of the stresses which they apply, are:—

1. Pressure (rolling).
2. Impact (ramming).
3. Vibration.

Types of Compaction Plant

9·64 The types of compaction plant that are available can be listed under these three headings as follows:—

(1) Rollers:— Smooth-wheel, pneumatic-tyred, sheepsfoot, lorries and pneumatic-tyred construction plant and track-laying vehicles.

(2) Rammers:—Dropping weight (including piling equipment), internal combustion type and pneumatic type.

(3) Vibrators:—Out-of-balance weight type and pulsating hydraulic type.
(Either of these types may be mounted on screeds, plates or rollers).

Factors affecting the Field Compaction of Soil

9·65 The factors affecting the field compaction of soil are similar to those influencing the compaction of soil in laboratory tests. The more important factors are the moisture content of the soil and the amount of compaction applied with the particular plant.

9·66 At the Road Research Laboratory investigations have been made under controlled conditions to determine the effect of these factors on the performance of some of the better-known types of compaction plant. Five soils ranging from a heavy clay to a gravel-sand-clay were used. The particle-size distributions and index properties of these soils are given in Fig. 9·19 and the maximum dry densities and optimum moisture contents obtained with laboratory compaction tests are given in Table 9·6. The investigations were all made on the soil when thoroughly broken up to form a loose layer 9 in. thick. The compaction obtained with the plant was determined by measuring the dry density of the compacted soil by the sand-replacement method using holes 6 in. deep, except in the case of the sheepsfoot rollers where 4-in. holes were used.

Smooth-wheel Rollers

9·67 The smooth-wheel rollers used for soil compaction include the conventional three-wheel type which weighs from 30 cwt. to 18 tons, tandem rollers ranging in weight from 1 to 14 tons and three-axle tandem rollers weighing from 12 to 18 tons. On some types of smooth-wheel roller the distribution of weight between the rolls can be adjusted by ballasting the rolls with water or by means of a heavy sliding weight.

9·68 The principal characteristics of smooth-wheel rollers affecting their performance in compacting soil are the load per inch width under the compaction rolls, and the width and diameter of the rolls. The load per inch width and the diameter control the pressure in the surface layer of the soil, while the dimensions of the rolls affect the rate with which this pressure decreases with depth. Thus, for compaction work, it is important to specify the load per unit width as well as the gross weight of a smooth-wheel roller. The performance of smooth-wheel rollers in compacting soil is illustrated by the results of tests made with an 8-ton smooth-wheel roller and a 2¾-ton smooth-wheel roller (Plate 9·6). The leading dimensions and loadings of the rollers are given in Table 9·5.

Soil	Liquid limit-per cent	Plastic limit-per cent	Plasticity index-per cent
Heavy clay	75	28	47
Silty clay	43	24	19
Sandy clay	27	19	8
Sand	Non-plastic		
Gravel-sand-clay	Non-plastic		

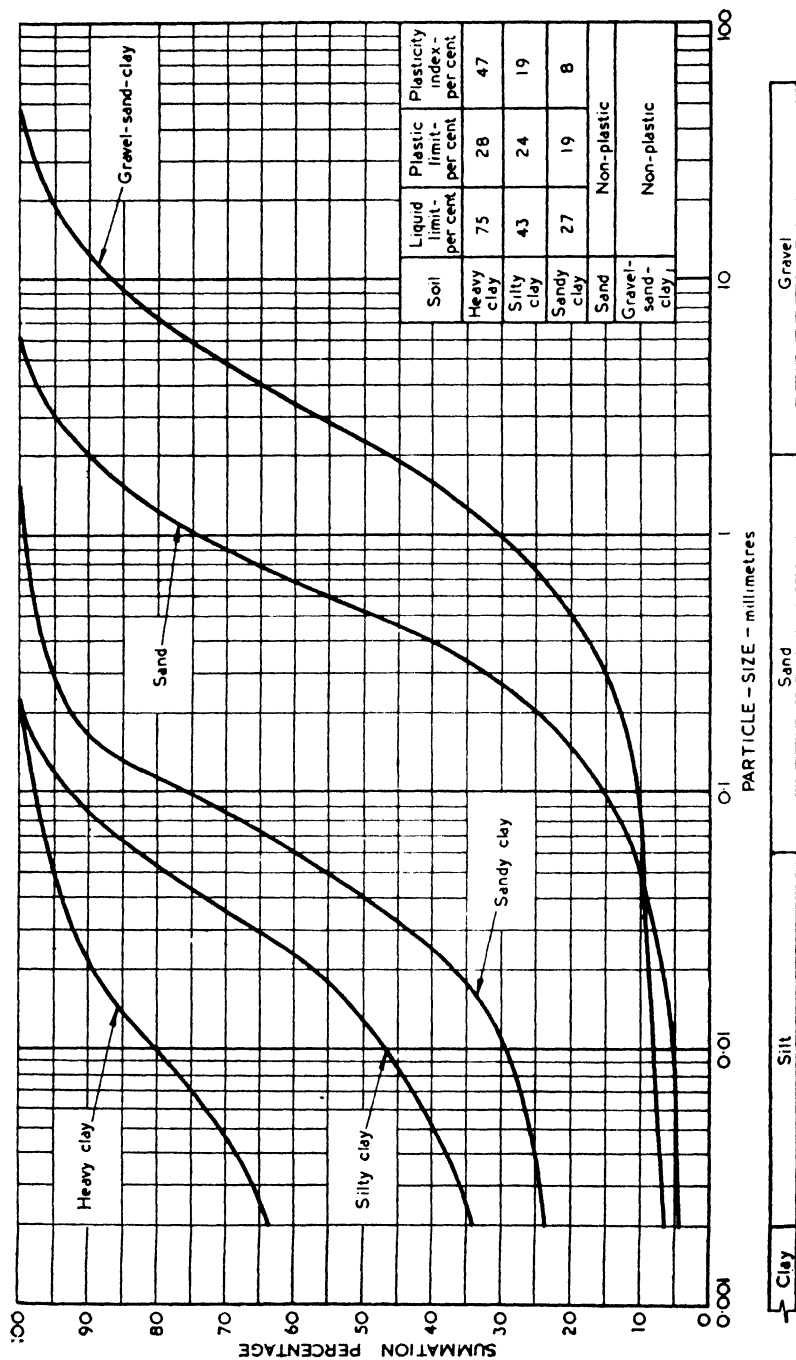



FIG. 9·19 INDEX PROPERTIES AND PARTICLE-SIZE DISTRIBUTION OF SOILS USED IN COVERED TRACK

TABLE 9·5

CHARACTERISTICS OF THE 8-TON SMOOTH-WHEEL ROLLER AND THE 2¾-TON SMOOTH-WHEEL ROLLER

	8-ton roller	2¾-ton roller
Gross weight	19,012 lb.	6,160 lb.
Weight on front rolls	7,822 lb.	1,904 lb.
Weight on rear rolls	11,190 lb.	4,256 lb.
Diameter of front rolls	42 in.	34 in.
Width of front rolls	21 in. (x2)	12 in. (x2)
Diameter of rear rolls	54 in.	36 in.
Width of rear rolls	18 in.	15 in.
Rolling width	70 in.	51 in.
Wheel base	9 ft 6 in.	6 ft
Load per in. width		
Front roll	186 lb.	80 lb.
Rear roll	311 lb.	142 lb.
$\frac{\text{Load (lb.)}}{\text{Width (in.)} \times \text{diameter (in.)}}$		
Front roll	4.4	3.9
Rear roll	5.8	2.3

9·69 The dry density/moisture content relations obtained on the five soils when compacted to refusal by these rollers are shown in Figs. 9·20, and 9·21, while the maximum dry densities and optimum moisture contents derived from these curves are compared with the corresponding values for well known laboratory compaction tests in Table 9·6. Referring to Table 9·6, the maximum dry densities obtained with the smooth-wheel rollers will be seen to be either approximately equal to, or in excess of, the corresponding values obtained in the B.S. compaction test. The performance of the rollers was best on the sand and gravel-sand-clay for which dry densities similar to those given by the modified A.A.S.H.O. tests were obtained. Apart from the case of the sandy clay the true optimum moisture content for the rollers was less than the corresponding value given by the B.S. compaction test and corresponded more closely with the values given by the Dietert test, where this was applicable.

9·70 It was found that the front and rear rolls of the 8-ton roller gave the same compaction. This is illustrated for the silty clay soil in Fig. 9·22 where the dry densities produced by the front and rear rolls using 6-in. and 4-in. density holes have been plotted against the moisture content. The front roll of the 2¾-ton roller, however, acted only as a spreader. From a consideration of the data given in Table 9·5 in relation to the compaction obtained with the rolls of different diameter, it would appear that the value of

$$\frac{\text{Load on roll (lb.)}}{\text{width of roll (in.)} \times \text{diameter of roll (in.)}}$$

gives a better indication of the performance of a roll for compacting soil than the load per in. width on a roll.

9·71 The effect of number of passes on the dry density of the five soils at their optimum moisture contents for compaction with the rollers is shown

in Fig. 9·23. The curves are characterized by a rapid increase in density for the first 8 passes of both rollers, followed in the case of the 2¾-ton roller by a slower subsequent increase in density. The gradual increase in density obtained after 8 passes of the 2¾-ton roller is considered to be due to the decrease in the contact area between the rolls and the ground as the soil becomes firmer,

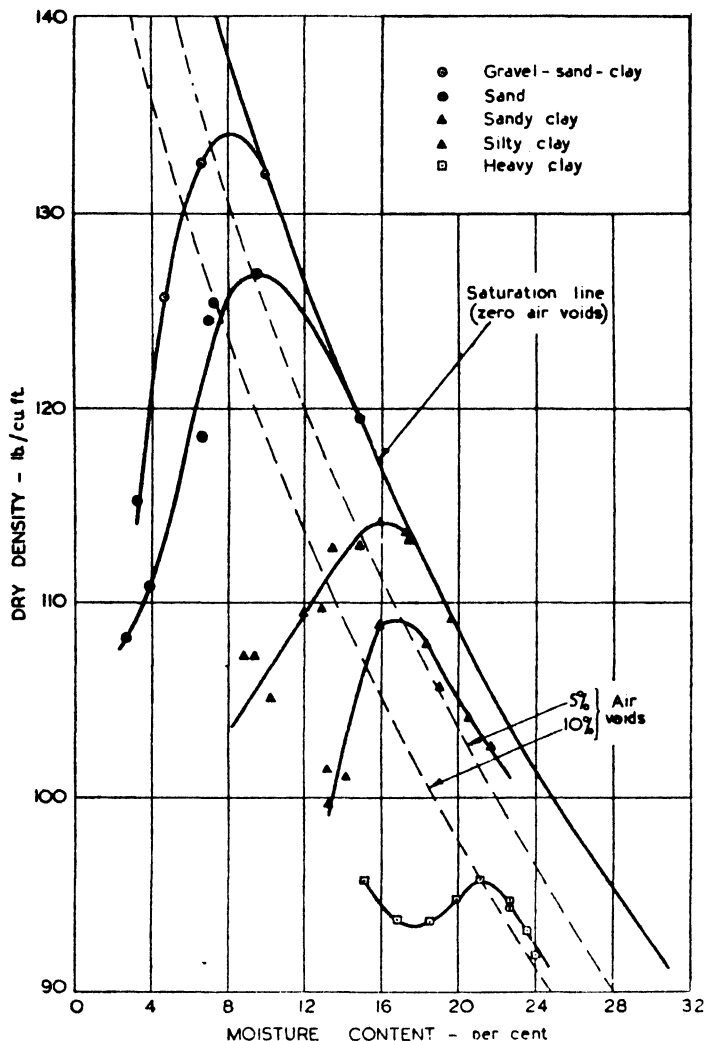



FIG. 9·20 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF A 2¾-TON SMOOTH-WHEEL ROLLER

causing an increased pressure on the soil. In the case of the 8-ton roller little increase in dry density was obtained after 8 passes and a reasonable degree of compaction was obtained after 8 passes of the 2¾-ton roller. The small increase in compaction obtained for further passes with this roller is probably not justified economically.

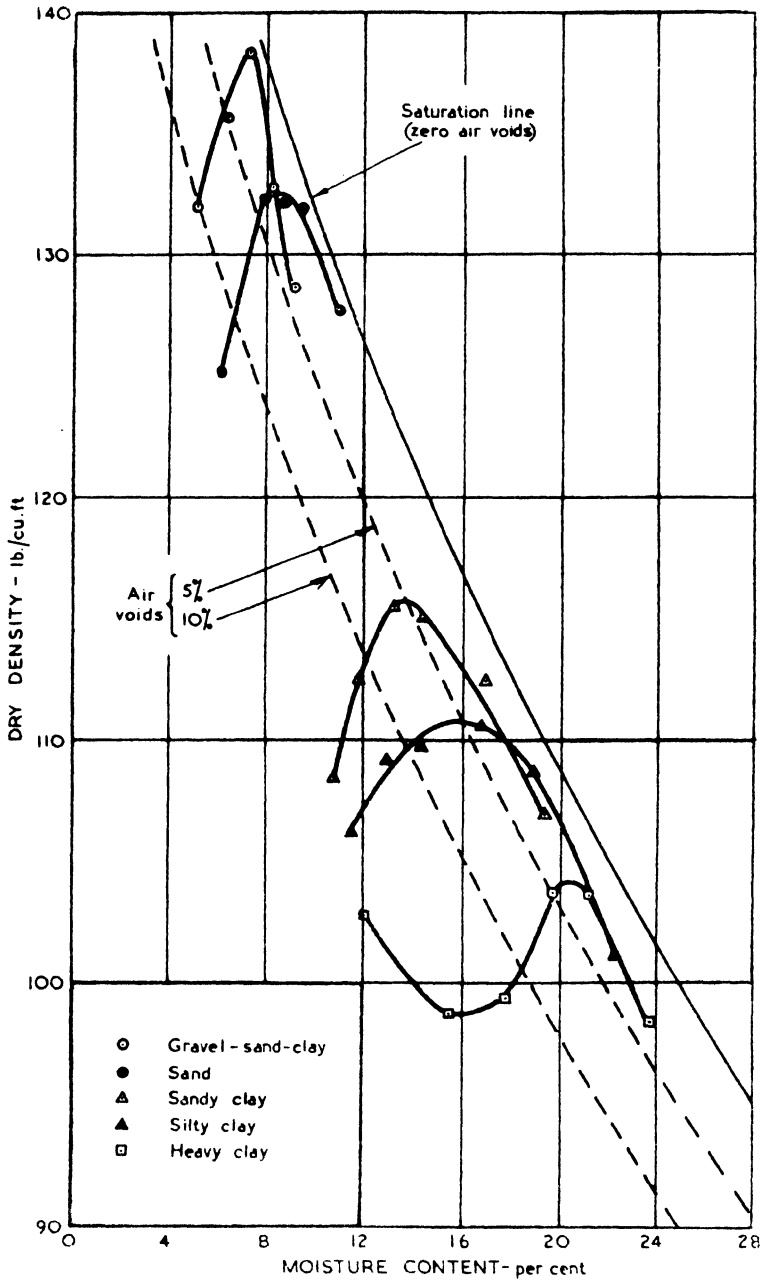



FIG. 9·21 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 32 PASSES OF AN 8-TON SMOOTH-WHEEL ROLLER

9·72 The effect of differences in the moisture content of soil on the relation between dry density and number of passes of smooth-wheel rollers is illustrated for the silty clay by the curves obtained with the 8-ton roller (Fig. 9·24).

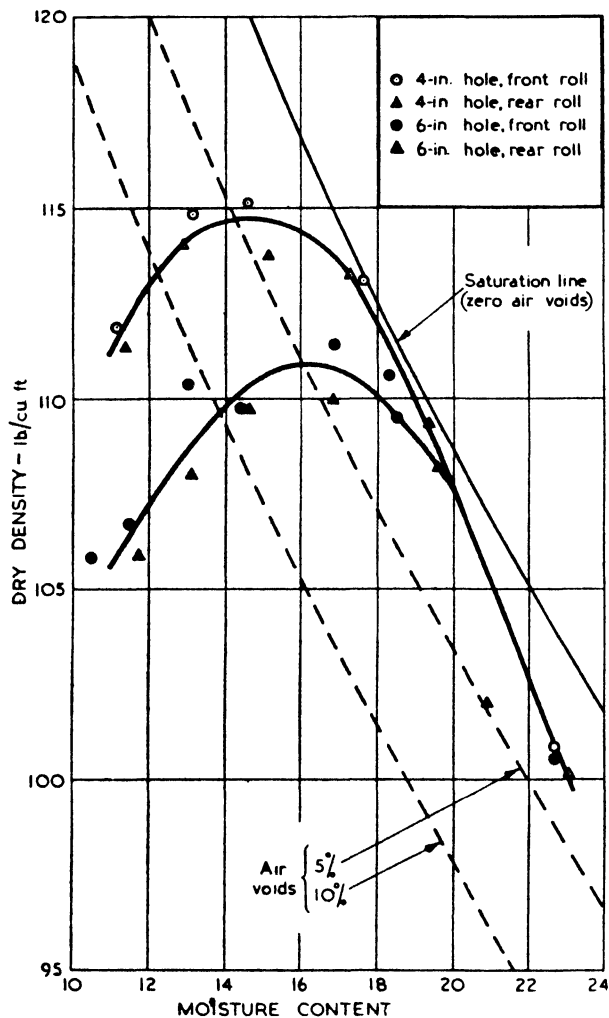



FIG. 9·22 COMPARISON OF THE RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR THE SILTY CLAY SOIL WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 32 PASSES OF THE 8-TON SMOOTH-WHEEL ROLLER, FRONT AND REAR ROLLS, USING 6-IN. AND 4-IN. DENSITY HOLES

9·73 On the basis of these results and other experience with smooth-wheel rollers, it has been found that they are most suitable for compacting gravels, sands, hardcore, crushed rock and any material where a crushing action is needed. The control of moisture content is critical in the case of mechanically stable gravels and sands, for which it should be adjusted to the optimum moisture content of the B.S. compaction test. The depth of layer compacted

depends on the weight of the roller and the purpose of the work, but will in general vary from about 6 in. for subgrades to 18 in. for the base of embankments.

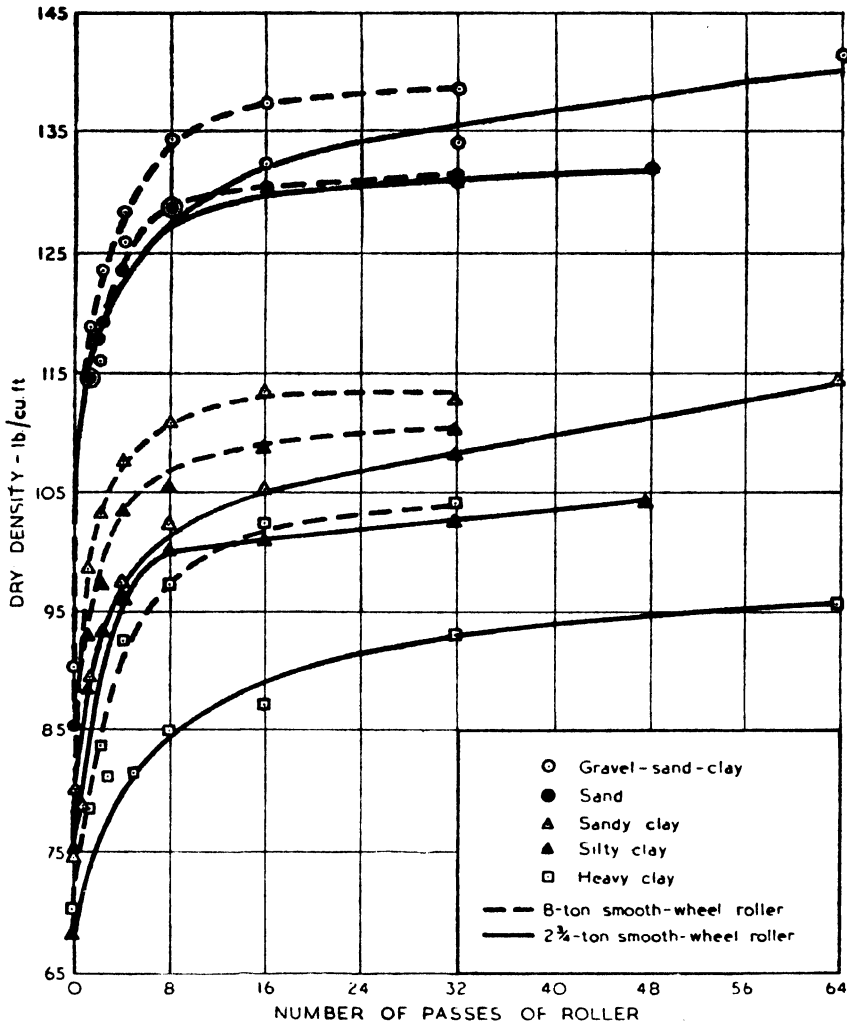



FIG. 9·23 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 8-TON AND 2¾-TON SMOOTH-WHEEL ROLLERS FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENTS FOR ROLLER COMPACTION

Pneumatic-tyred Rollers

9·74 A common form of pneumatic-tyred roller consists of a box or platform mounted between two axles, the rear of which has one more wheel than the front, the wheels mounted on the front axle being arranged to track in between those mounted on the rear axle. One type of roller in use in this country has

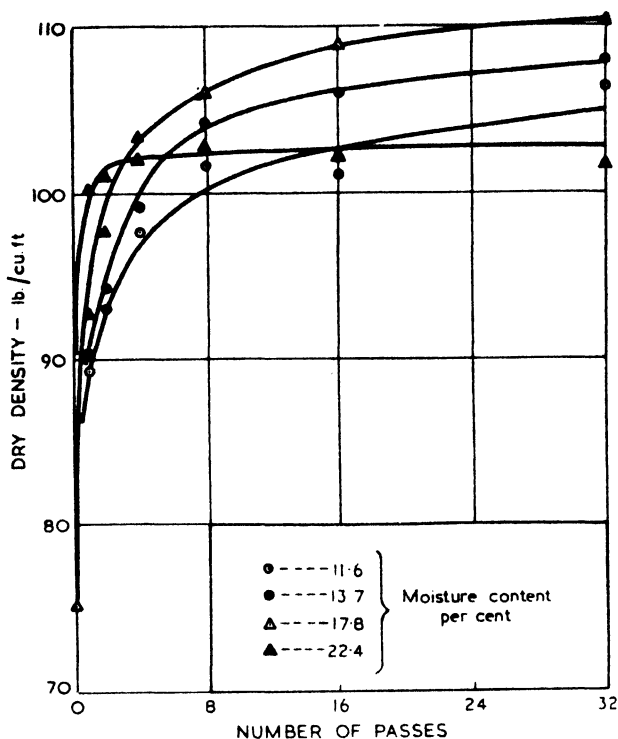



FIG. 9·24 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 8-TON SMOOTH-WHEEL ROLLER FOR THE SILTY CLAY SOIL WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT DIFFERENT MOISTURE CONTENTS

four front wheels and five rear wheels, but the actual number of wheels varies according to the make and gross weight of the roller. One type of pneumatic-tyred roller known as the " wobble-wheel roller " has wheels mounted at a slight angle with respect to the axle; this provides a kneading action which is claimed to give improved results.

9·75 Pneumatic-tyred rollers are loaded with kentledge such that when the tyres are inflated to their desired pressure the sum of the contact widths of the tyres approximately equals 80 per cent of the width of the roller. The rollers are normally towed by either a track-laying or a pneumatic-tyred tractor.

9·76 The characteristics of a pneumatic-tyred roller affecting its performance in compacting soil are the tyre inflation pressure and the area of contact between the tyre and the ground. The gross weight of a pneumatic-tyred roller is a secondary factor since it is a function of the size and inflation pressure of the tyres and the number of wheels. The behaviour of this type of roller is illustrated by the results of tests made with a 12-ton roller having nine wheels and a tyre inflation pressure of 36 lb./sq.in. (Plate 9·7). The dry density/moisture content relations obtained on the five soils when compacted to refusal by this roller are shown in Fig. 9·25, while the maximum dry densities and optimum

moisture contents derived from these curves are compared with the corresponding values for well known laboratory compaction tests in Table 9·6.

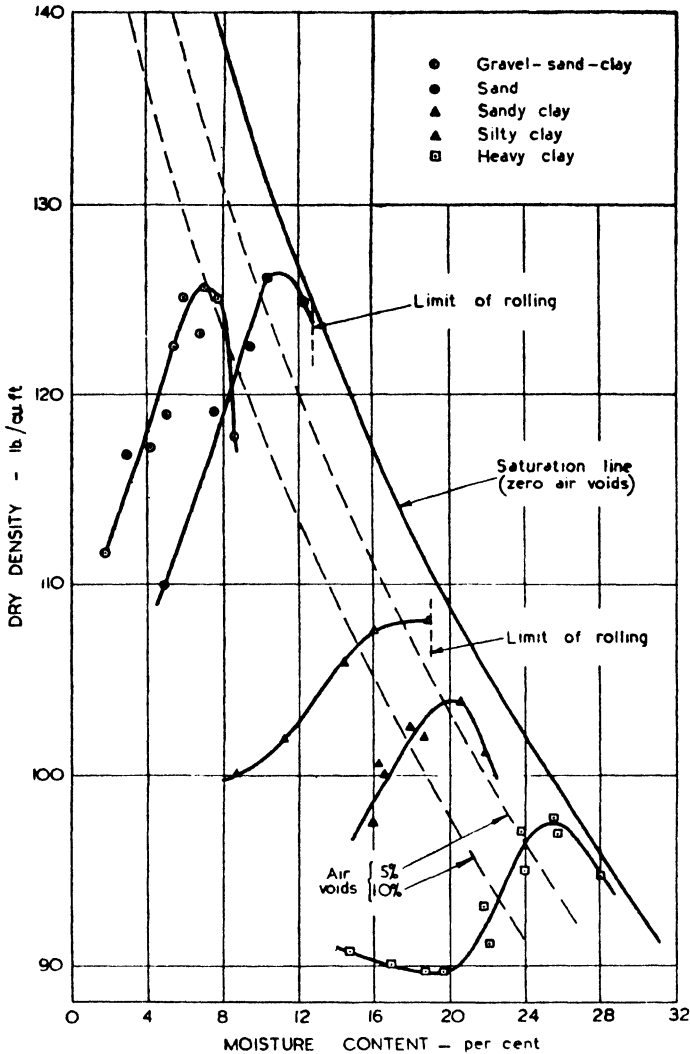



FIG. 9·25 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF A 12-TON PNEUMATIC-TYRED ROLLER

9·77 For all the soils except the sandy clay, the values of maximum dry density obtained with the roller were either approximately equal to, or in excess of, the corresponding values obtained with the B.S. compaction test. Its performance on the sand was particularly good. Except for the sandy clay soil the values of optimum moisture content for compaction with the roller agreed closely with the values given by the B.S. compaction test.

TABLE 9·6

COMPARISON OF MAXIMUM DRY DENSITIES AND OPTIMUM MOISTURE CONTENTS OBTAINED WITH COMPACTION PLANT AND IN LABORATORY TESTS

Soil type	Heavy clay		Silty clay		Sandy clay		Sand		Gravel-sand-clay	
Place of origin	Staines, Middlesex		Laboratory grounds		Laboratory grounds		Hertingfordbury, Hertfordshire		West Drayton, Middlesex	
	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)	Maximum dry density (lb./cu.ft)	Optimum moisture content (%)
B.S. compaction test	97	26	104	21	115	14	121	11	129	9
Modified A.A.S.H.O. compaction test	113	17	120	14	128	11	130	9	138	7
Dietert compaction test ...	102	23	109	17	116	14	119	11	—	—
2¾-ton smooth-wheel roller ...	95	21	110	17	114	16	127	10	134	8
8-ton smooth-wheel roller ...	104	20	111	16	116	14	132	8	138	7
Pneumatic-tyred roller	98	25	104	20	108	19	127	11	126	7
Sheepsfoot roller (club-foot type)	107	16	116	14	119	12	—	—	129	6
Sheepsfoot roller (taper-foot type)	107	15	115	14	120	12	—	—	128	5
½-ton frog-rammer	107	17	110	15	116	13	128	10	136	7

9·78 The effect of number of passes on the dry density of the soils when compacted at or just above their optimum moisture content by the roller are shown in Fig. 9·26. As in the case of the smooth-wheel roller only 8 passes were, in general, required to obtain a dry density almost equal to the value that could be ultimately obtained for a very large number of passes. In the case of the sand and sandy clay soil the pneumatic-tyred roller was used to study the effect of the moisture content of the soils on the relation between the dry density and number of passes of the roller. The results of these tests are given in Figs. 9·27 and 9·28. It was found that within the range of moisture contents

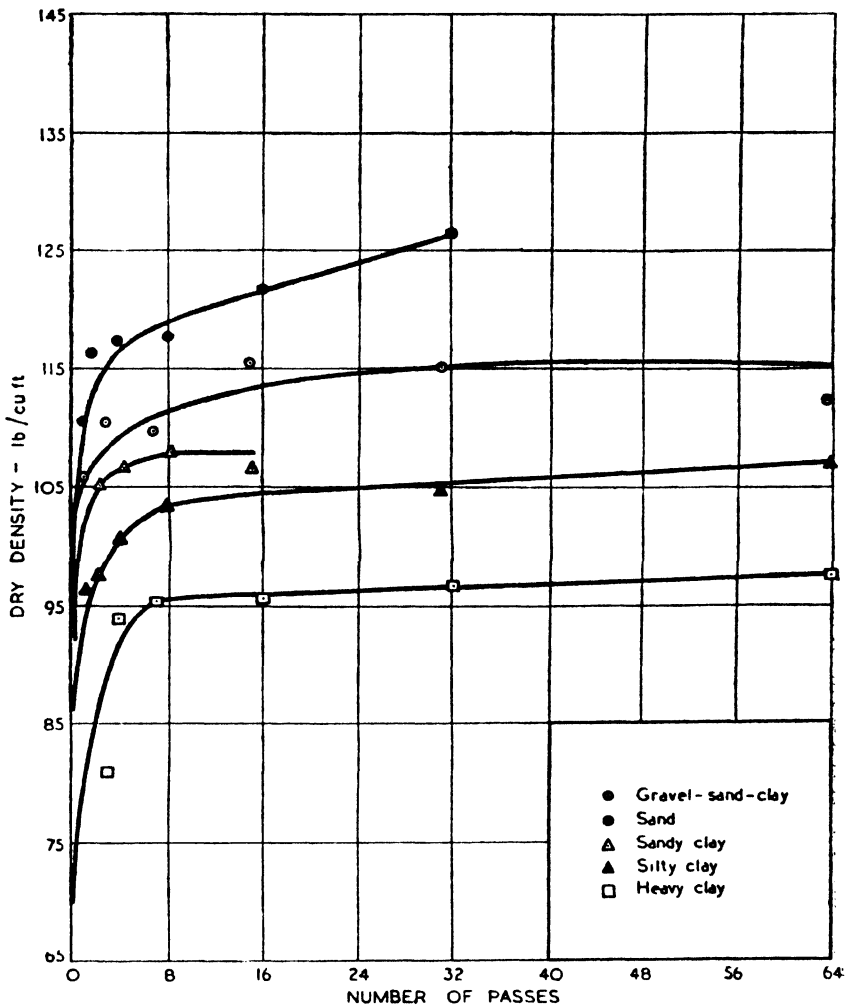



FIG. 9·26 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE PNEUMATIC-TYRED ROLLER FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENT FOR ROLLER COMPACTION

at which rolling was possible with these soils, the number of passes required to obtain the highest dry density at any given moisture content decreased as the soil moisture content increased. In practice this means that the smallest number of passes was required when the soils were either at or a little above their optimum moisture contents. There will, of course, be cases where soils can be compacted several per cent above their optimum moisture content; although in such cases fewer passes would be necessary at the higher moisture content to reach the highest dry density obtainable, this dry density might be less than that obtainable by compacting the soil at the optimum moisture content.

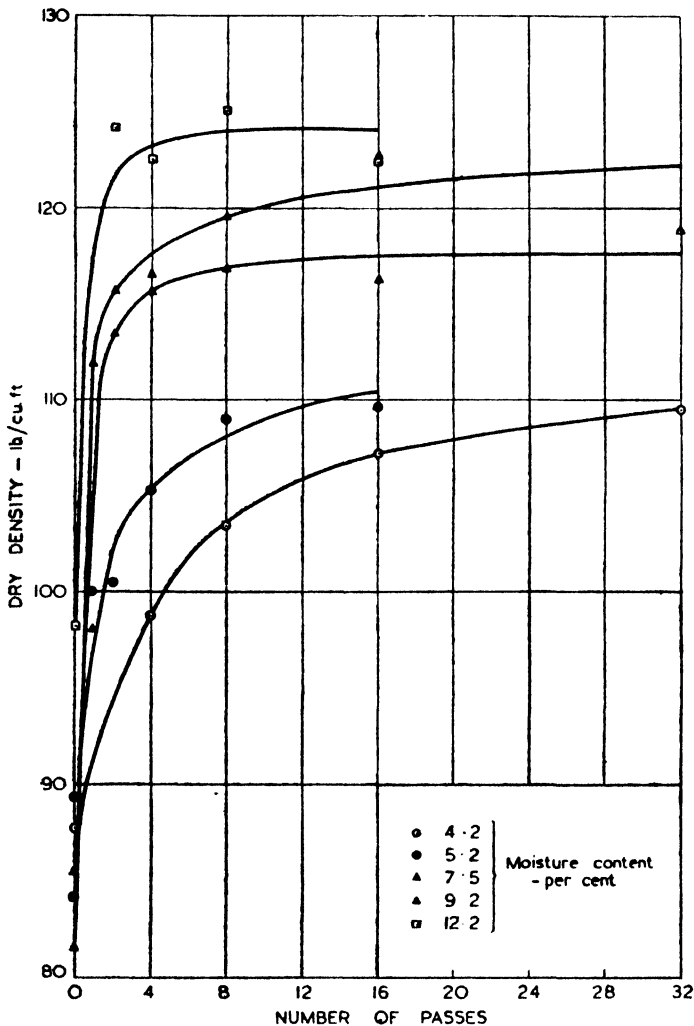



FIG. 9·27 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 12-TON PNEUMATIC-TYRED ROLLER FOR A SAND WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT DIFFERENT MOISTURE CONTENTS

9·79 Summarizing the above results, pneumatic-tyred rollers appear to be most suitable for compacting fine-grained soils and, in particular, closely graded sands. For cohesive soils the rollers give their best performance when the soil is about 2-4 per cent below the plastic limit. The optimum moisture content will, however, decrease as the load and tyre pressure are increased. The depth of layer should not, in general, exceed 9 in. and should be less than this value where good compaction is essential.

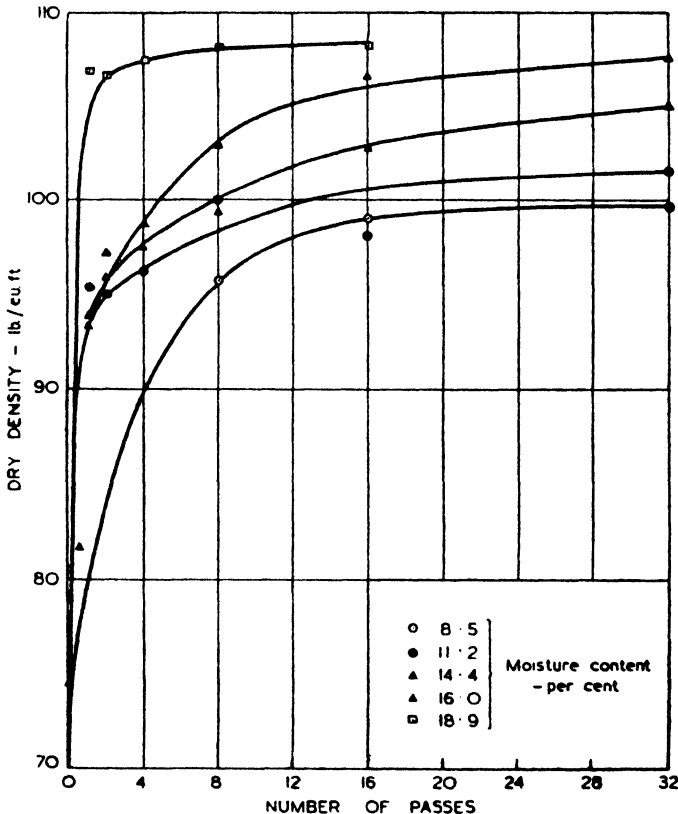



FIG. 9·28 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 12-TON PNEUMATIC-TYRED ROLLER FOR A SANDY CLAY WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT DIFFERENT MOISTURE CONTENTS

Construction Traffic

9·80 The effectiveness of construction traffic in compacting soils is similar to that of pneumatic-tyred rollers. However, owing to the fact that the tyres do not give complete coverage with each pass and to the tendency of drivers to follow existing tracks, the soil is usually well compacted under the wheel tracks, leaving badly compacted areas between them. When the rollers are subsequently used they are then not able to compact these soft areas since

they ride on the well compacted tracks. Care should therefore be taken, when using construction traffic for compacting fills, to ensure that the wheel tracks are spread over the entire surface.

Sheepsfoot Rollers

9·81 Sheepsfoot rollers consist of a hollow cylindrical steel drum on which projecting feet are mounted. The drums can be ballasted either with water or with wet sand and they are mounted either singly or in pairs on a welded steel frame. Various makes of sheepsfoot roller are available having different diameters and widths of drum and different lengths and shape of feet. The most common type available in the British Isles is one having two drums 4 ft wide and 3 ft 6 in. in diameter with feet 8 in. long; according to the shape of the feet the rollers may be described either as taper-foot or club-foot rollers. Sheepsfoot rollers are towed by either track-laying or pneumatic-tyred tractors; up to three rollers may be towed by a heavy tractor.

9·82 The principal characteristics of sheepsfoot rollers affecting their performance in compacting soil are the foot pressure and the coverage of ground obtained per pass. These in turn depend upon the gross weight of the roller, the area of each foot, the number of feet in contact with the ground at any time and the total number of feet per drum.

9·83 To illustrate the performance of sheepsfoot rollers, results are given of investigations made with a club-foot roller having a gross weight of 5 tons, 4 x 3-in. feet, and a foot pressure of 115 lb./sq.in. (Plate 9·8A) and a taper-foot roller having a gross weight of 4½ tons, 2¼ x 2¼-in. feet, and a foot pressure of 250 lb./sq.in. (Plate 9·8B). The dry density/moisture content relations obtained when four soils were compacted by 64 passes of each of the rollers are shown in Figs. 9·29 and 9·30 and the maximum dry densities and optimum moisture contents derived from the curves are given in Table 9·6. The sheepsfoot rollers were unable to compact the sand, the properties of which are given in Fig. 9·19. Referring to Table 9·6, it will be noted that the two rollers give almost identical values of maximum dry density and optimum moisture content. The values of optimum moisture content obtained agreed most closely with the corresponding values given by the modified A.A.S.H.O. compaction test and were considerably lower than those given by the B.S. compaction test. On the three cohesive soils the maximum dry densities obtained with the rollers exceeded those obtained with the B.S. compaction test, but were less than those given by the modified A.A.S.H.O. compaction test. The curves relating dry density and number of passes for the two types of sheepsfoot roller (Figs. 9·31 and 9·32) were similar for the silty and sandy clays and for the gravel-sand-clay, but in the case of the heavy clay the club-foot roller gave a dry density approaching the ultimate value obtainable after 32 passes, whereas the taper-foot roller, judging from the shape of the curve, had still not reached a final dry density after 64 passes. In general, the results indicated that a further increase in dry density would have been obtained with additional passes in excess of 64, but apart from exceptional circumstances where conditions demand very high compaction it is probably not practicable to apply more than 24 passes. Apart from the case of the club-foot roller on the heavy clay soil the dry density of the soil after 24 passes was of the order of 10 per cent less than that after 64 passes.

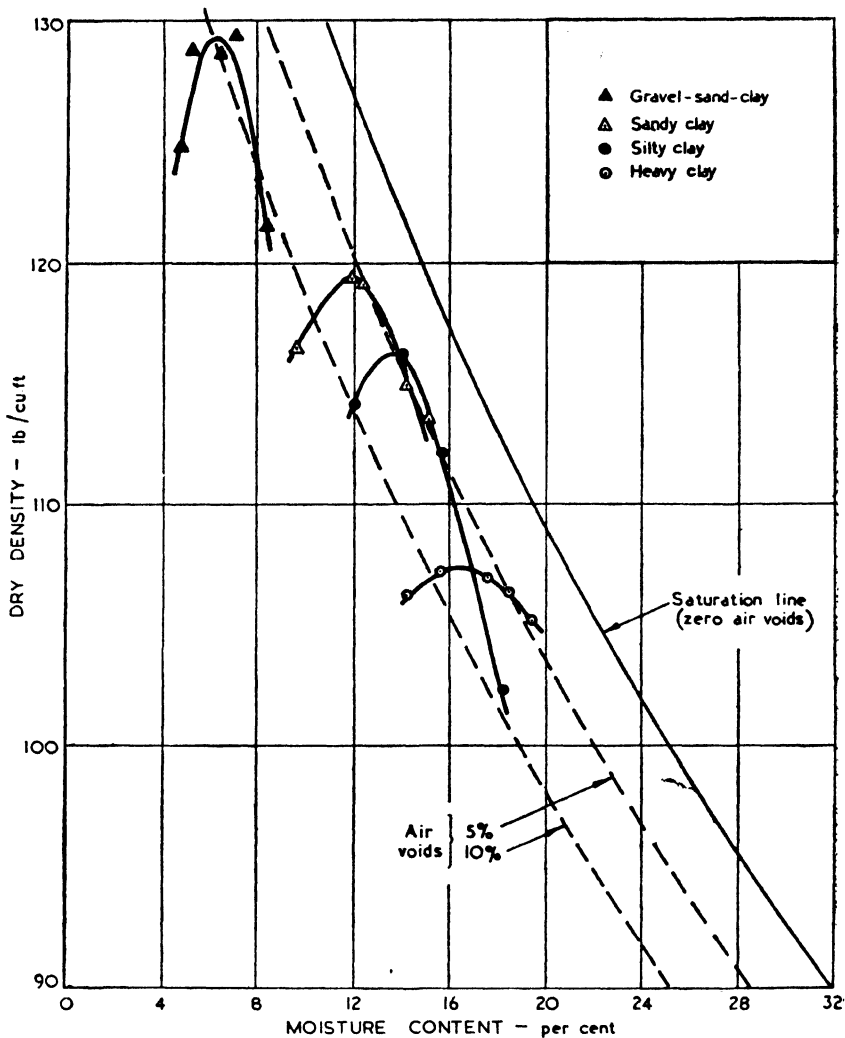



FIG. 9·29 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FOUR DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF A 5-TON CLUB-FOOT SHEEPSFOOT ROLLER

9·84 The effect of the moisture content of the soil on the relationship between dry density and number of passes was studied in the case of the taper-foot roller compacting the cohesive soils. Typical results in this investigation are illustrated in Fig. 9·33 for the heavy clay soil; these show that the optimum moisture content varied from about 21 per cent for 8 passes or less to 16 per cent for 64 passes. At the higher moisture content fewer passes were required to reach the maximum dry density obtainable at that moisture content.

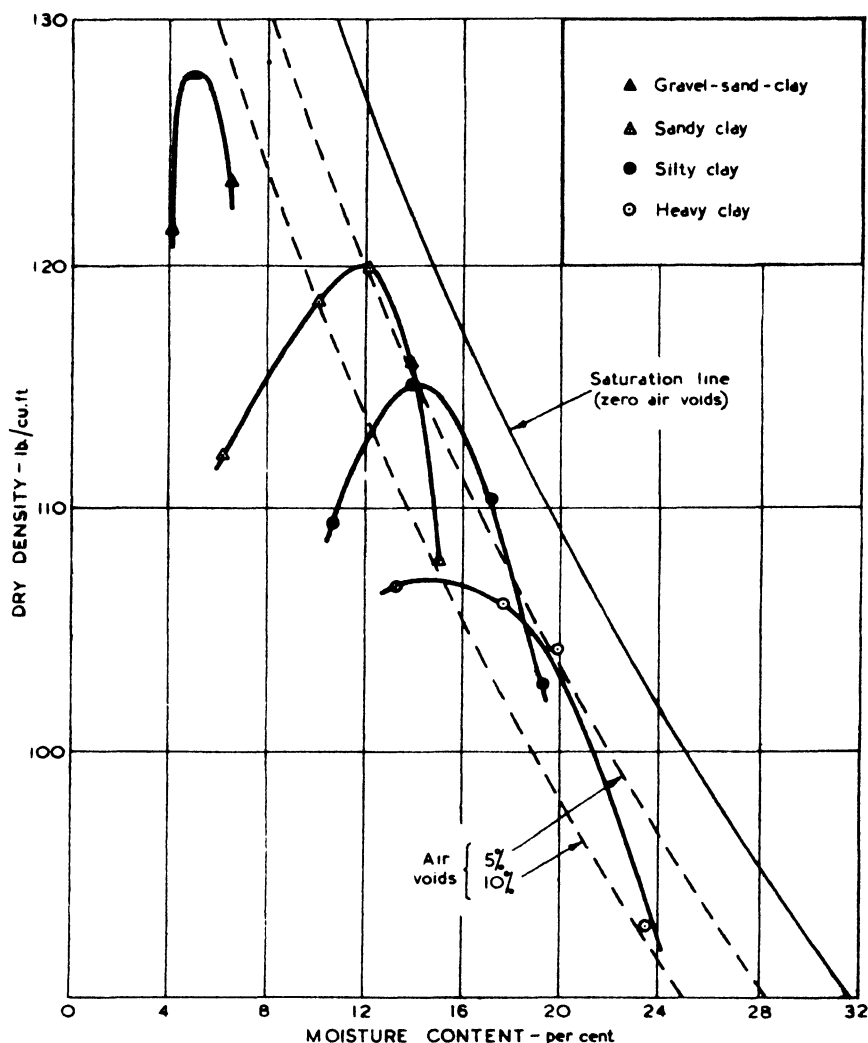



FIG. 9·30 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FOUR DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF A 4½-TON TAPER-FOOT SHEEPSFOOT ROLLER

9·85 Summarizing the results of these tests, sheepsfoot rollers of the type tested appear to be most suitable for compacting cohesive soils in which case the moisture content should be approximately the same as the optimum moisture content given by the modified A.A.S.H.O. compaction test. These moisture contents are lower than are usually found in practice in the British Isles. At least 24 passes are necessary with the rollers to obtain reasonably adequate compaction, and they should not be used to compact a layer of soil more than 2 in. greater than the length of their feet. As shown in Figs. 9·29 and 9·30 soil compacted by sheepsfoot rollers has a rather greater air voids content than

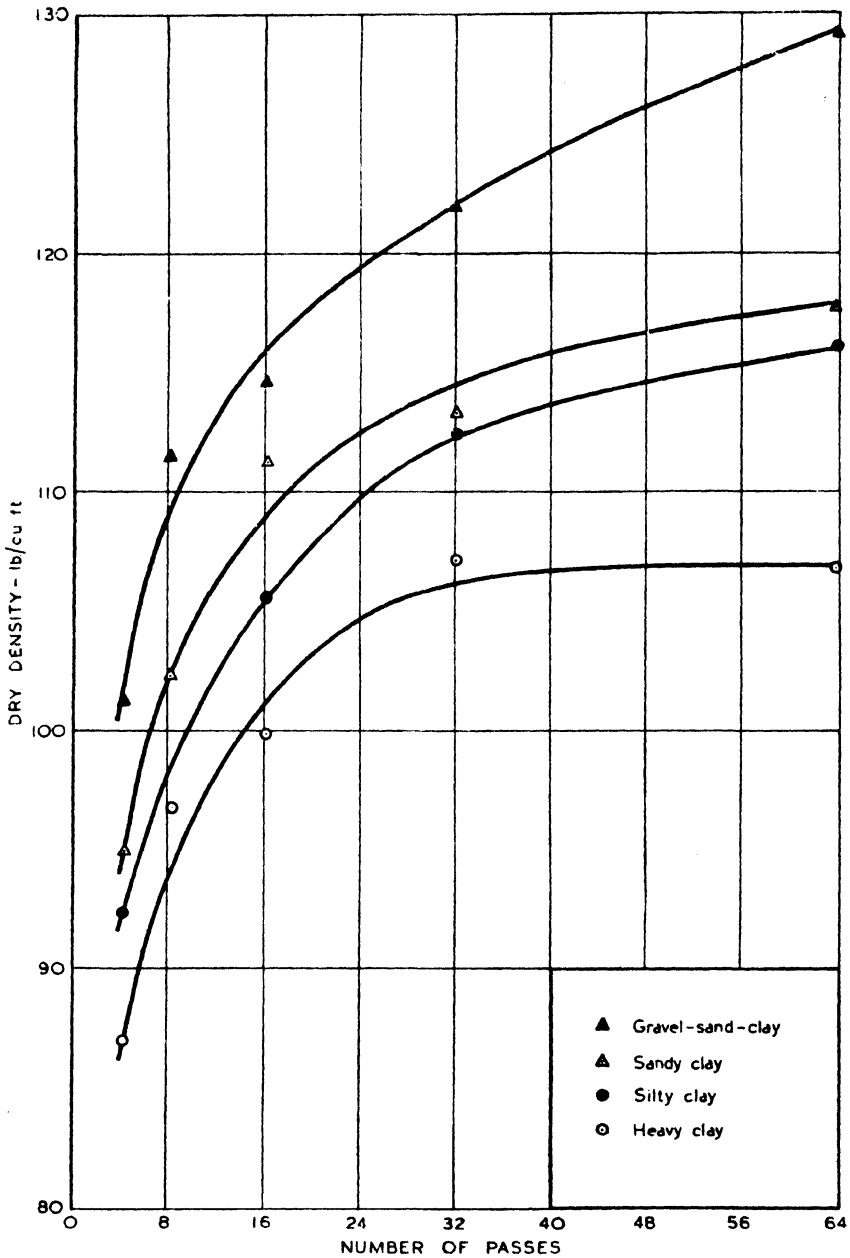



FIG. 9·31 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 5-TON CLUB-FOOT SHEEPSFOOT ROLLER FOR FOUR DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENTS FOR ROLLER COMPACTION

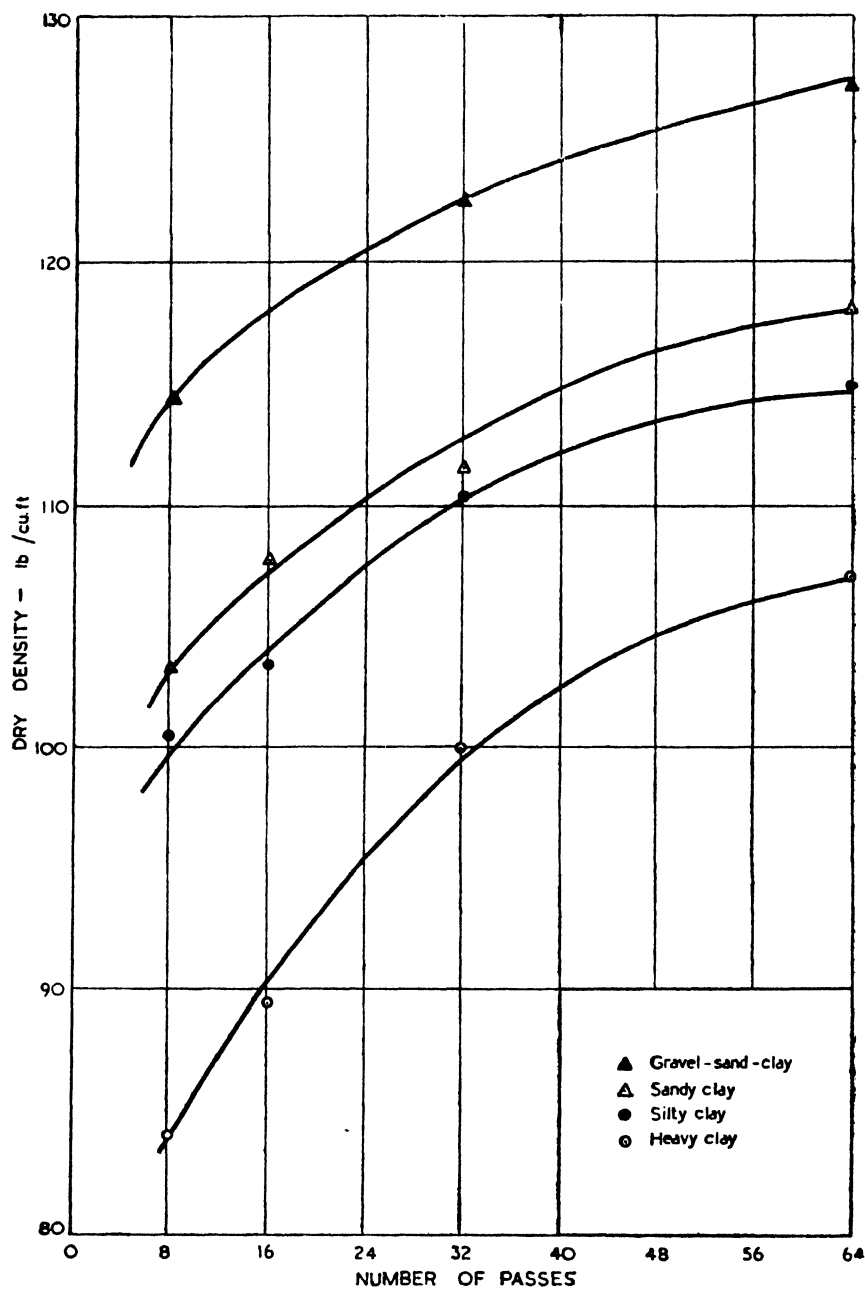



FIG. 9·32 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 4½-TON TAPER-FOOT SHEEPSFOOT ROLLER FOR FOUR DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENTS FOR ROLLER COMPACTION

soil compacted by smooth-wheel or pneumatic-tyred rollers, and this may be deleterious in the case of subgrades for which a large air content increases their liability to absorb water.

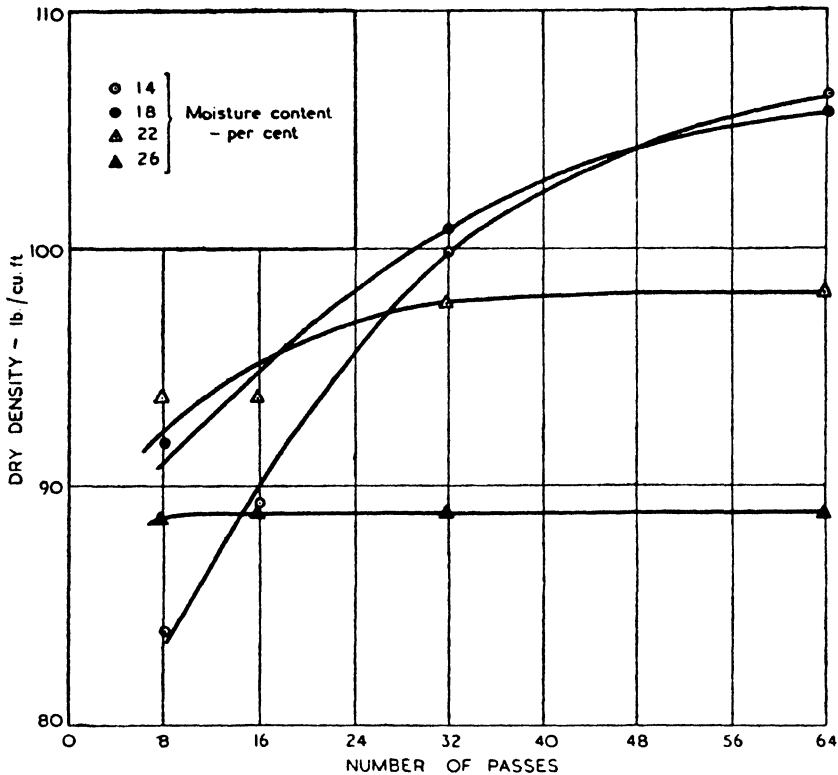



FIG. 9·33 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF PASSES OF THE 4½-TON TAPER-FOOT SHEEPSFOOT ROLLER FOR A HEAVY CLAY WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT DIFFERENT MOISTURE CONTENTS

Vibrators

9·86 Vibrators consist of a vibrating unit of either the out-of-balance weight type or a pulsating hydraulic type mounted on a screed, plate or roller. Investigations at the Laboratory have shown that good compaction can be obtained with vibrators on cohesionless soils. Tests have been made with a vibrating screed (Plate 9·9A), a vibrator mounted on two types of plate (Plate 9·9B) and a vibrator mounted on a 4½-cwt. hand-propelled roller (Plate 9·10A).

9·87 The dry density/moisture content relations obtained with those types of plant on sandy soils are compared with the corresponding curves obtained with the B.S. compaction test in Figs. 9·34, 9·35 and 9·36. In all the cases examined it will be seen that the vibrators gave maximum dry densities much in excess of the corresponding value in the B.S. compaction test at optimum moisture contents several per cent below the standard compaction value.

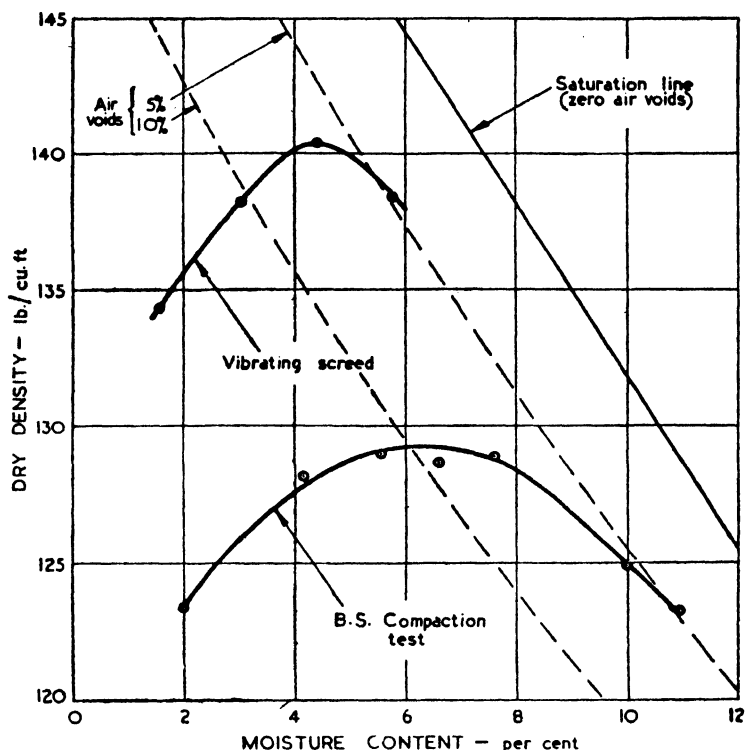



FIG. 9·34 DRY DENSITY/MOISTURE CONTENT CURVES FOR A GRAVEL-SAND SOIL OBTAINED WITH A VIBRATING SCREED AND IN THE B.S. COMPACTION TEST

9·88 The application of vibrators to the compaction of soil is still in an experimental stage and, owing to their very low output, their use is at present limited. Their main application appears to be to the compaction of cohesionless sands and gravel-sands where very high densities are required and compaction by otherwise suitable plant is prevented by limitations of space or other practical conditions.

Rammers

9·89 Rammers for compacting soil comprise pneumatic and internal combustion types weighing from 70 lb. to 3 cwt., internal combustion type frog-rammers weighing up to one ton, and dropping weights, all of which are capable of compacting soil to a satisfactory dry density. To illustrate the performance of rammers, results are given of investigations made with a ½-ton frog-rammer which had a base diameter of 2 ft 5 in. (Plate 9·10B). The dry density/moisture content relations obtained when the five soils were compacted by five coverages of the rammer are shown in Fig. 9·37 and the maximum dry densities and optimum moisture contents derived from the curves are given in Table 9·6. The table shows that the maximum dry densities and optimum moisture contents obtained on all the soils were between those given by the B.S. and modified A.A.S.H.O. compaction tests. The curves relating

dry density and number of coverages of the frog-rammer when compacting the five soils at or just above the optimum moisture contents for compaction with the rammer are given in Fig. 9·38. In general between 2 and 4 coverages were required to obtain a dry density approaching the maximum obtainable for a large number of coverages.

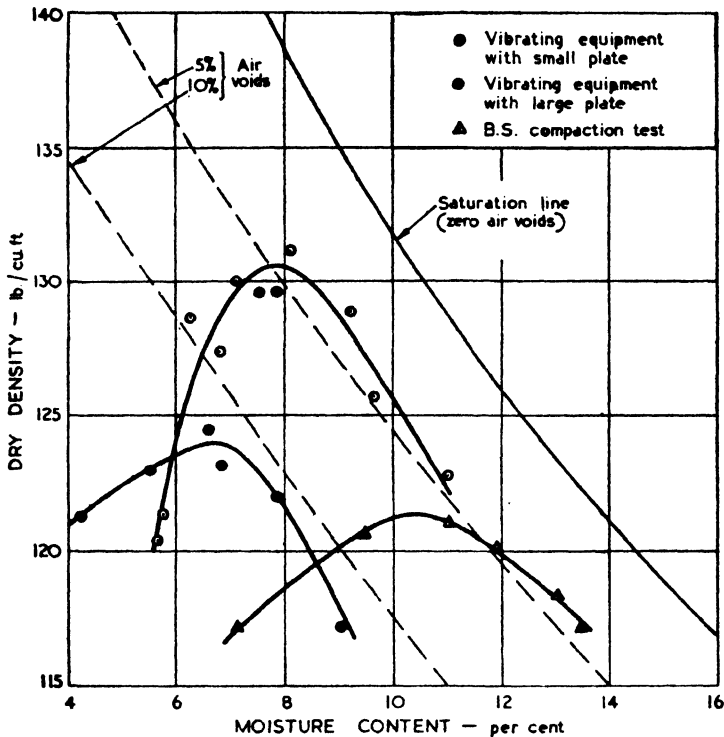



FIG. 9·35 DRY DENSITY/MOISTURE CONTENT CURVES FOR A SANDY SOIL OBTAINED WITH VIBRATING EQUIPMENT AND IN THE B.S.COMPACTION TEST

9·90 Tests were also made to determine the vertical gradient of dry density through the resulting compacted layer when 30-in. loose layers of the silty clay, the sand and the gravel-sand-clay were compacted by six coverages of the frog-rammer. For each soil, an average dry density equal to the maximum dry density in the B.S. compaction test was obtained; with the granular soils the density gradient was $\frac{1}{2}$ lb./cu.ft/in. depth of the compacted layer and 1 lb./cu.ft/in. depth for the silty clay soil.

9·91 The frog-rammer was found to be able to compact an area of 280 sq.yd per hour. Summarizing the above results, rammers of the type tested can compact relatively thick layers of non-cohesive soils, but the layers should not be thicker than 9 in. for clays. It is thought that the thickness of soil layer that can be satisfactorily compacted is dependent on the diameter of the base of the rammer. The moisture content of soil compacted by a heavy rammer should be maintained just below the optimum moisture content determined

by the B.S. compaction test. Owing to the low output of rammers their use is largely limited to special sites where rollers are not able to operate, such as for trenches and behind bridge abutments.

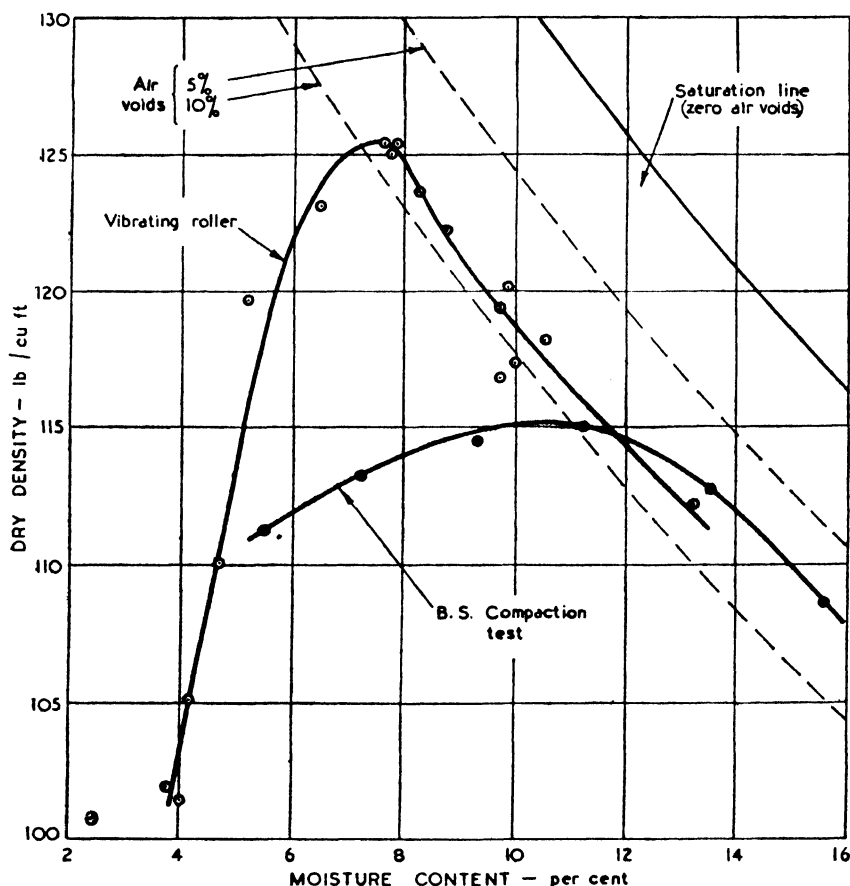



FIG. 9·36 DRY DENSITY/MOISTURE CONTENT CURVE FOR A SANDY SOIL OBTAINED WITH A VIBRATING ROLLER AND IN THE B.S. COMPACTION TEST

General Remarks on Field Compaction

9·92 The performance of compaction plant is dependent on the soil type, its particle-size distribution and its moisture content, and these factors must be taken into account in selecting compaction plant for a particular job. In general, smooth-wheel rollers are most suited to crushed rock, hardcore, mechanically stable gravels and sands, pneumatic-tyred rollers to closely graded sands and fine-grained cohesive soils at moisture content approaching their plastic limits, and sheepsfoot rollers to fine-grained cohesive soils at moisture contents of from 7 to 12 per cent below their plastic limits.

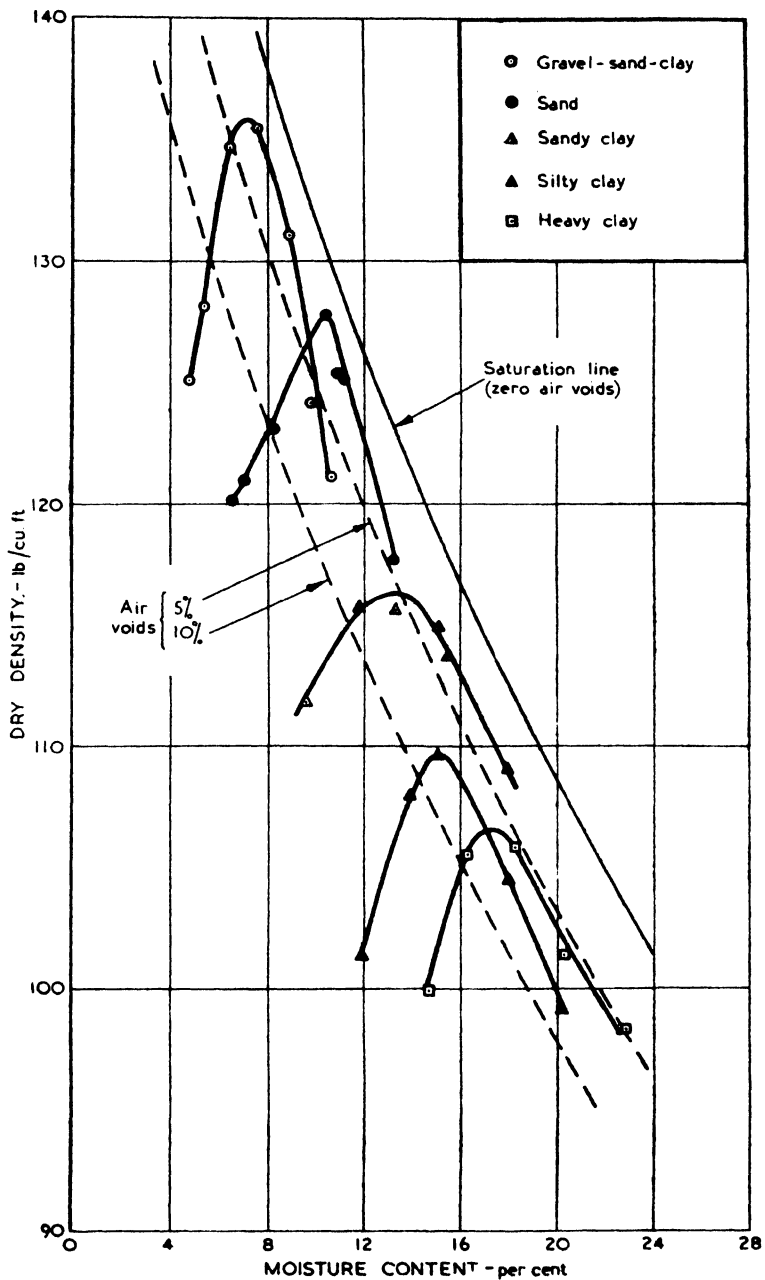



FIG. 9·37 RELATIONSHIPS BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 5 COVERAGES OF A ½-TON FROG-RAMMER

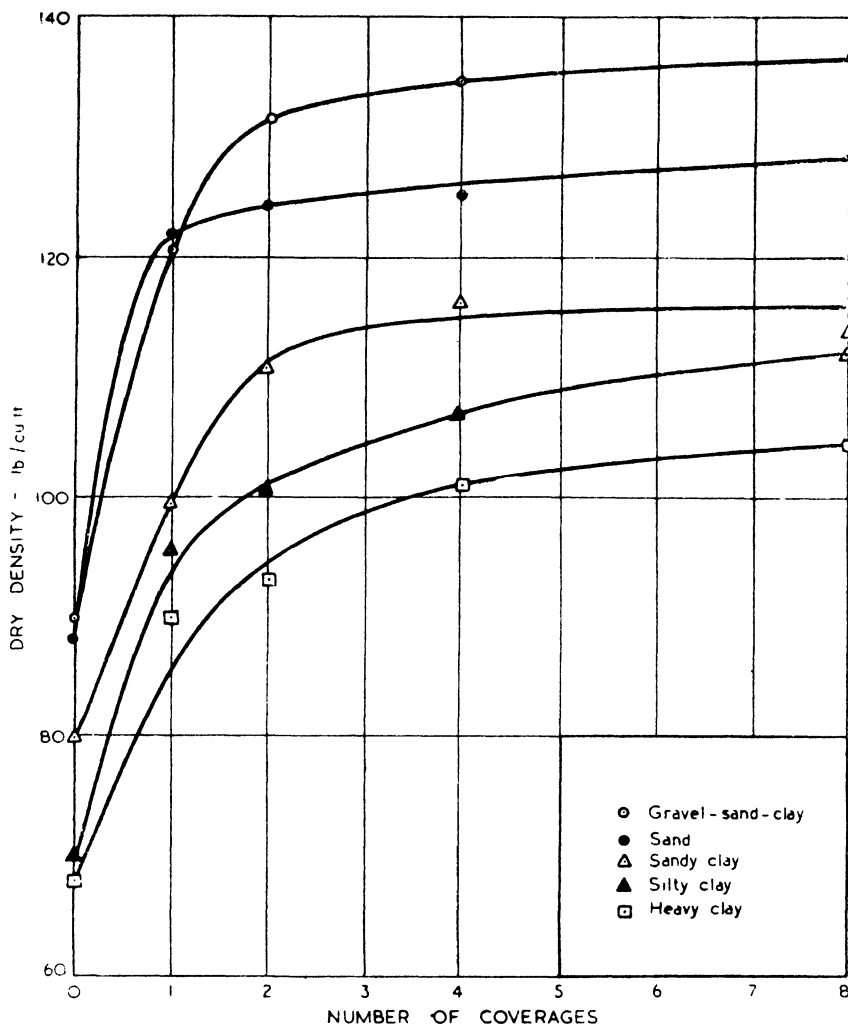



FIG. 9·38 RELATIONSHIPS BETWEEN DRY DENSITY AND NUMBER OF COVERAGES OF THE ½-TON FROG-RAMMER FOR FIVE DIFFERENT SOILS WHEN COMPACTED IN 9-IN. LOOSE LAYERS AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENTS FOR COMPACTION WITH THE FROG-RAMMER

9·93 The dry density of compacted soil decreases with depth as the thickness of the layer compacted is increased. With normal compaction plant, this reduction is not very great up to loose thicknesses of about 9 in. but above 9 in. it is considerable.

9·94 It has also been found that when thinner layers are used, the entrapped air can be driven out very much more quickly with a smaller amount of work. In experiments[10] using internal combustion and pneumatic types of rammers for compacting soils ranging from a heavy clay to a sand, it has been found

that their output can be doubled when the thickness of layer is reduced from 8 in. to 4 in.

9·95 The effect of admixtures is chiefly of interest in stabilized soil construction. Hogentogler[11], who has investigated the effects on cohesive soils of numerous admixtures, found that in general the admixture of electrolytes increases the maximum density by from 5 to 10 per cent and also decreases the optimum moisture contents. Calcium chloride, which is used to improve gravel roads in dry climates, has been found to increase the dry density by as much as 11 per cent. The effect of admixtures such as Portland cement, bituminous and resinous materials upon the maximum dry density and optimum moisture content is described in Chapters 12, 13 and 14.

Relative Compaction and Efficiency of Compaction

9·96 The degree of compaction obtained in the field is measured by the " relative compaction," that is the ratio of the field dry density to the maximum dry density obtained in the B.S. compaction test expressed as a percentage. The relative compaction of a loose soil is usually about 75 to 80 per cent and that of a well compacted soil about 100 per cent.

9·97 In order to obtain a better scale on which to compare the efficiency of compaction of various types of plant, there should be a zero corresponding to the loose density of the soil, and the upper limit should correspond to a high degree of compaction, say 100 per cent relative compaction.

9·98 A convenient formula for the efficiency of compaction is therefore:—

$$\frac{\text{Field dry density} - \text{Loose dry density}}{\left.\begin{array}{l}\text{Maximum dry density}\\ \text{obtained in B.S.}\\ \text{compaction test}\end{array}\right\} - \text{Loose dry density}} \times 100 \text{ per cent}$$

Suggested Procedure for carrying out Full-scale Field Compaction Trials

9·99 When earthworks are constructed in the field it is important that some form of control of compaction be maintained during construction to ensure that a satisfactory degree of stability is obtained. The degree of control will vary according to the size and nature of the job, and each job must be treated on its merits.

9·100 On a small job, sufficient control will be obtained if it is ensured that the soil is spread in 9-in. layers, given 8 passes of a smooth-wheel roller and sprayed with water if necessary so that the soil is wet enough just to ball in the hand. On large jobs stricter control is advisable, and this will usually entail working to a specified degree of compaction. In this case field trials may lead to the saving of a considerable amount of money by ensuring that the most economical procedure for spreading and compacting the soil is determined.

Technique for Determining the Correct Procedure for Compacting Earthworks

9·101 PRELIMINARY INVESTIGATIONS. As a preliminary to the main field trials, it is desirable to undertake the following tests on the available filling materials:—

(1) B.S. compaction test.
(2) Identification tests, e.g. the liquid and plastic limit tests for cohesive soils or the particle-size analysis for non-cohesive soils.

9·102 As already indicated, the B.S. compaction test only gives a guide to the optimum moisture content to be used in the field, but it enables the best of several filling materials to be chosen, the best being the material giving the highest maximum dry density.

9·103 The purpose of the identification tests is to classify the soil and hence to select the most suitable type of available plant for compacting the soil.

9·104 FIELD TRIALS. A test area 20 yd long by 15 yd wide is prepared on the actual site of construction from which the top-soil has been removed. The fill material to be used is spread over this area in three strips 5 yd wide, the depth of the loose material in the strips being varied over the range of thicknesses it is desired to study. Normally, the range of values investigated would be from 6 to 18 in. No adjustment should be made to the moisture content of the fill ma erial which, apart from minor fluctuations, should be in its natural moisture condition.

9·105 The test area is then compacted with the plant it has been decided to use and the mean dry density of the full depth of each strip determined after 2, 4 and 8 passes for all types of plant except for sheepsfoot rollers, for which the measurements of dry density should be made after 4, 8 and 16 passes. The dry densities of the soil should be determined by either the core-cutter or sand-replacement method, whichever is most suitable, and it is recommended that the mean of five determinations should be obtained for each soil condition.

9·106 This procedure should be repeated, if possible, at two other moisture contents, the values suggested being the optimum moisture content given by the B.S. compaction test and a moisture content intermediate between this value and the natural moisture content of the soil. If the natural moisture content of the soil is similar to the optimum moisture content, it is suggested that the additional tests be made at moisture contents 3 per cent on either side of this value. These tests should be made on strips of fresh fill material laid on an adjacent area from which the top-soil has been removed.

9·107 The test procedure has been described in detail, but it can usually be considerably shortened by noting the trend of the early results, and by the experience of the engineers carrying out the tests. These trials would take about a week to complete, and could be started as soon as the necessary plant arrives on the site, while the preliminary clearing operations are in progress. The trials can thus be completed before the main construction starts. From the results of the trials and a knowledge of the costs of the various procedures, it is possible to determine the most economical procedure to be adopted to obtain the specified degree of compaction, and the subsequent control work can be simplified.

Application of Suggested Procedure

9·108 An example is given where this procedure was undertaken. To illustrate the procedure, in connexion with the construction of chalk embankments, the compaction of the chalk by a heavy smooth-wheel roller was investigated on the site.

9·109 The chalk was fairly soft (upper chalk) and contained a small percentage of flint. The chalk lumps when dug were as much as 2 ft in size, but these were broken down to about 6 in. before compaction.

Preliminary Tests

9·110 B.S. compaction tests were carried out on samples of the chalk. Using material passing a ¾-in. B.S. sieve, it was impossible to obtain a well defined dry density/moisture content curve, but a satisfactory curve was obtained with material passing the No. 7 B.S. sieve. A maximum dry density of 100 lb./cu.ft was obtained at an optimum moisture content of 23 per cent.

9·111 The moisture content of the chalk lumps was found to remain almost constant at 25 per cent and it was decided not to consider the effect of varying the moisture content. This was borne out on the site, because in spite of heavy rain during the investigation the moisture content of chalk lumps remained within $\pm$ 2 per cent of the mean value of 25 per cent.

Field Trials

9·112 The roller used was a heavy smooth-wheel steam roller having a gross weight of 11½ tons, giving a load per inch width of 430 lb. for the rear rolls.

9·113 In the tests on the site, the test layer was spread to the required thickness either on the natural chalk from which the overburden had been removed, or on chalk which had previously been compacted. In both cases, the test layer of uncompacted chalk was compacted by 1, 2, 4 and 8 passes of the roller and determinations of the dry density were made at each stage. In all the tests on the site, the determinations of dry density were made on areas compacted by the rear wheels.

9·114 The core-cutter method was used to determine the dry density throughout the work, as it was found to be the most suitable method. The density of the undisturbed chalk was 92 lb./cu.ft.

Results of Tests

9·115 NUMBER OF PASSES. The dry density of the chalk has been plotted against number of passes of the roller for 6-in. and 9-in. layers of chalk spread (1) by hand and (2) by an angle-dozer (Fig. 7·3). In all cases, only a small increase in dry density was obtained after 4 passes.

9·116 THICKNESS OF LAYER. Fig. 7·3 shows that for 4 passes of the heavy roller the dry density of the hand-spread 6-in. layer was about 2½ lb./cu.ft higher than that of the corresponding 9-in. layer; in the case of chalk spread by the angle-dozer the corresponding difference was about 3½ lb./cu.ft.

9·117 METHOD OF SPREADING. As shown in Fig. 7·3, a valuable increase in the dry density from 60 to 84 lb./cu.ft was obtained by running the tracks of the angle-dozer over the chalk prior to rolling. The final compaction after rolling was also much higher. This effect appeared to be due to the crushing action of the track plates together with the vibration of the machine. The compacting effect of the angle-dozer was observed to operate to a depth of 5 in., and the effect would not be so valuable for layers of chalk greater than this.

9·118 INTERPRETATION OF RESULTS. It was suggested that the material should be spread at its natural moisture content in 6-in. layers by an angle-dozer, and rolled with 4 passes of a heavy smooth-wheel roller. In view of the uniformity of compaction obtained on the trial strips, it was also suggested that the control should be limited to seeing that this procedure was observed.

SUMMARY

9·119 Soil compaction is the process whereby soil is mechanically compressed through a reduction in the air voids. In road construction, good compaction is needed in the building of embankments and for subgrades, bases and sub-bases. In an embankment, subsequent settlement can be minimized, thus enabling a permanent road structure to be placed on it immediately after its completion. Compaction of a subgrade increases its stability and resistance to water absorption.

9·120 The compaction of soil is measured in terms of its dry density or the amount of solid matter in a cubic foot. Soils range in dry density from about 140 lb./cu.ft for coarse-grained gravels and sands to about 90 lb./cu.ft for heavy clays. For a constant amount of compaction, all soils have an optimum moisture content at which a maximum dry density is obtainable. At a constant moisture content, increased amounts of compaction increase the dry density of the soil until the air voids are almost eliminated or the resistance of the soil to further compaction becomes too great.

9·121 Descriptions are given of laboratory tests used to determine the compaction characteristics of soil and factors affecting the results of these tests are discussed. The effect of compaction on the strength of soil is illustrated by studies made of the California bearing ratio and unconfined compressive strength of laboratory-remoulded soils.

9·122 Four field methods of measuring the dry density of compacted ground are described, and factors affecting their use are discussed; a rapid field method of measuring the moisture content of soil with the Proctor needle is also described.

9·123 In the field, compaction is effected by different types of roller, rammer or vibrator. Trials have been made at the Laboratory of the compaction of five British soils by smooth-wheel, pneumatic-tyred and sheepsfoot rollers and vibrating plant. Dry density/moisture content relations similar to those for laboratory compaction tests were obtained in these trials. The optimum moisture contents obtained with the pneumatic-tyred roller were similar to the values obtained in the B.S. compaction test, but the sheepsfoot rollers, the smooth-wheel rollers (on the sand and the gravel-sand-clay) and vibrators gave optimum moisture contents similar to those given by the modified A.A.S.H.O. compaction test. With the remainder of the equipment tested, viz. smooth-wheel rollers on cohesive soils and a ½-ton frog-rammer, the optimum moisture contents were between the values given by the B.S. and modified A.A.S.H.O. compaction test. In general, dry densities equalled or exceeded the maximum dry densities obtained with the B.S. compaction test, when the soil was at the optimum condition for compaction.

9·124 Studies were also made of the relation between dry density and number of passes of the plant. With smooth-wheel and pneumatic-tyred rollers about eight passes were required before a dry density approaching the highest obtainable was reached; with the sheepsfoot rollers a further increase in dry density could in general be obtained after 64 passes.

9·125 A procedure is outlined for determining the most economical combination of type of plant, soil moisture content, number of passes and thickness of layer for obtaining the required compaction on large earthworks and the application of this procedure to the construction of an embankment is described.

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CHAPTER 10

ROAD CONSTRUCTION WITH SOIL AND LOW-GRADE AGGREGATES

INTRODUCTION

10·1 The earliest form of highway consisted of an earth track which had been cleared of vegetation and compacted by the human and animal traffic which used it. In later years these tracks were widened and had gravel or broken stone spread on them to enable them to carry early forms of vehicular traffic. It is only in recent times that networks of " metalled," all-weather roads have been developed in the more thickly populated areas of the world, and even today the earth road predominates in Africa, Asia and elsewhere. Recently, the need for cheap, but at the same time efficient, roads in undeveloped areas has resulted in the technical development of earth road construction in which use has been made of soil and low-grade forms of aggregate. This development has taken place principally in the U.S.A. but also in the British Dominions and Colonies. This chapter reviews these technical developments and deals briefly with the principles of construction.

GENERAL CONSIDERATIONS

10·2 As with modern forms of road, the construction of an earth road comprises the preparation of the subgrade followed by the construction of a base and the laying of a surfacing. However, the surfacing is sometimes omitted for cheapness, and the base may be formed by the special treatment of the soil composing the subgrade. The essential feature of earth road construction is thus the preparation of the base using either soil or poor aggregate as the main constructional material.

10·3 The earth road can also be regarded as the first phase in the construction of a modern highway by forming the foundation of a future high-grade form of road employing either a bituminous surfacing or concrete and designed to carry heavy traffic. This development of the road structure in step with traffic requirements is often referred to as " stage " construction.

SUBGRADES

10·4 It is important that the subgrade of an earth road should have adequate stability. Peat and highly organic soils should be avoided or excavated and replaced with more stable material. In areas where severe frosts occur, frost-susceptible soils, such as silts and chalks, should be excluded from the frost zone either by excavating them and replacing with non-susceptible material or by placing a sufficient thickness of such material on top of the subgrade as fill. In either case the imported material would also function as the base. In areas where considerable seasonal changes in moisture content occur, the

(A) SAND-CLAY ROAD, SOUTHERN NIGERIA

(B) GRAVEL-SAND-CLAY ROAD, CALIFORNIA

PLATE 10·1

(A) LATERITE ROAD, CENTRAL NIGERIA

(B) SAND-MIX COUNTRY ROAD, FIFESHIRE

PLATE 10·2

(A) SURFACE-DRESSED SOIL-CEMENT COUNTRY ROAD, SURREY

(B) SOIL-CEMENT ROAD WITH TARMACADAM SURFACING, BERKSHIRE

PLATE 10·3

P.B.S. ROAD WITH TARMACADAM SURFACING
Road Research Laboratory

PLATE 10·4

resulting volume changes in clay subgrades (such as those composed of " cotton soil ") can be minimized to some extent by rolling in granular material.

10·5 To ensure adequate stability the subgrade should be thoroughly compacted when the soil is at a favourable moisture content. Reference should be made to Chapter 9 for further information on soil compaction. The subgrade must not be allowed to become too wet during preparation, and practical measures, such as providing a suitable camber and filling depressions and ruts, should be taken to prevent rain-water standing on the formation.

10·6 The measures necessary to provide adequate drainage of the subgrade are similar to those for high-grade roads described in Chapter 17. It is particularly necessary to provide an impermeable surfacing or base to prevent rain-water reaching the subgrade. Ditches, in addition to removing surface water, can also assist with the drainage of the subgrade. In the U.S.A. ditches with a gentler slope from the road (Fig. 10·1) are being increasingly used. These can be easily maintained with a blade-grader, are less dangerous to traffic than the ordinary type and are less liable to erosion.
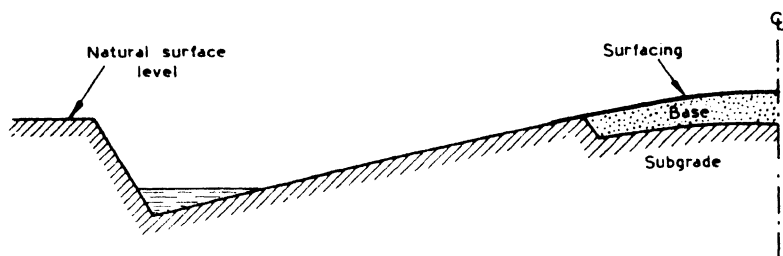
FIG. 10·1 SECTION OF DRAINAGE DITCH FOR SOIL ROAD

BASES

10·7 The base of an earth road usually comprises a stable soil or low-grade aggregate with or without an admixture of cement, bitumen or other stabilizing agent. In addition to the quality of the base material, the stability of the base is dependent on its compaction and thickness.

Stable Base Materials

10·8 A stable base material contains suitable proportions of gravel, sand, silt and clay. Occasionally, deposits of such material occur naturally, e.g. the desert and estuarine sand-clay deposits in Egypt and Nigeria, and the wall-ballasts and hoggins of the Thames valley and Hampshire basin. More often it is necessary to combine two or more soils in suitable proportions, a process sometimes referred to as mechanical stabilization. This subject is dealt with in detail in Chapter 11. Typical unsurfaced sand-clay and gravel-sand-clay roads are illustrated in Plate 10·1A and Plate 10·1B respectively.

10·9 A good base material can also be obtained by rolling low-grade aggregates, such as slag and limestone, which crush to form a dense mass with few large voids. A disadvantage of limestone is its susceptibility to frost. In the tropics various types of ironstone and laterite are extensively used for constructing bases. An unsurfaced lateritic road is shown in Plate 10·2A.

Soil Stabilization for Base Construction

10·10 Most soils, when well compacted and at a suitable moisture content, have satisfactory load-bearing properties but become unstable if their moisture content is increased. The object of soil stabilization is to maintain the soil in a state of high stability, or in some cases to increase the stability. It may be defined as " any process aimed at maintaining or improving the performance of a soil as a constructional material, usually by the use of admixtures." It will be noticed that this definition covers the use of soil for a wide range of engineering purposes, such as the construction of roads, airfields, buildings and irrigation works.

10·11 The materials employed as additives in soil stabilization have usually been the conventional road engineering binders, such as Portland cement and bituminous materials of various types. Use has also been made of industrial wastes and by-products such as calcium chloride and sulphite lye. Some of the development work with these materials has been done in North America, where the rapid development of the territory and geographical conditions have favoured the use of low-cost forms of road construction. Some idea of the extent and relative proportions to which they have been used can be gathered from a report by Mills[1] to the Highway Research Board of America. Table 10·1 gives data, based on his figures, for the mileages of various types of stabilized soil roads constructed in the U.S.A. between 1925 and 1939.

TABLE 10·1

LENGTH OF STABILIZED SOIL ROADS CONSTRUCTED IN THE U.S.A. BETWEEN 1925 AND 1939 (MILLS)

Type		Total miles
Soil of controlled composition		8551
Bituminous materials	Asphaltic bitumen 2763	3651
	Tar 572	
	Emulsion 316	
Cement		249
Chemicals		1917

10·12 A similar survey has not yet been made of construction carried out in the U.S.A. subséquent to 1939, but it is clear from the literature that the number of miles of stabilized soil laid will by now have increased considerably. In particular, extensive use was made of stabilized soil during the recent war for airfield construction in the U.S.A. and a summary of practice at that time was given by Markwick and Keep[2]. The use of soil stabilization has not been neglected in Great Britain where by 1950, over 700,000 sq. yd were known to have been used in connexion with roads and airfields.

10·13 In the present chapter only a classification of the various methods is given, stating the general characteristics of the materials produced. For convenience, these methods can be classified into two main groups, viz:—

(1) Materials which act as binding agents.

(2) Materials which stabilize the moisture content of the soil.

(1) Materials which act as Binding Agents

10·14 A large number of materials have been suggested for use as binding agents for soils. Of these, only three types have a practical value in the majority of cases. They are:—

(a) Cement.

(b) Bituminous materials.

(c) Sodium silicate.

10·15 In cement stabilization a small proportion of cement is mixed into the natural soil, which is then compacted at the optimum moisture content. After curing, which is generally done under humid conditions, the mixture sets in 7 to 28 days to give a material that is hard and reasonably resistant to the disintegrating effects of water and frost. Soil-cement has been successfully employed by several local authorities on housing estate schemes where economy in materials and labour and rapidity of construction were required[3] [4]. Considerable areas have also been laid in Africa in recent years for road and airfield construction. Examples of soil-cement roads are given in Plate 10·3 A and B. The stabilization of soil with cement is dealt with in detail in Chapter 12.

10·16 In bituminous soil stabilization a variety of materials such as asphaltic and cut-back bitumens, oils, tars and emulsions are added to the soil in small quantities, to bind the particles together. In the British Isles, such processes are at a disadvantage in so far as they involve increasing the fluids content of the soil. Under the prevailing climatic conditions, this is already fairly large for most of the year, so that the addition of a fluid binder tends to make cohesive soils plastic and difficult to compact. This does not apply in the case of sands, however, and an illustration of successful sand-bitumen construction, on a country road in Scotland, is shown in Plate 10·2B. In dry climates, the liquid nature of the binder is usually an advantage, since it provides part of the optimum fluids content for compaction, thereby economizing in water. The bituminous stabilization of soil is described in more detail in Chapter 13.

10·17 The use of sodium silicate in soil stabilization is due to its ability to react in aqueous solution with soluble calcium salts, forming insoluble and gelatinous calcium silicates. The chemical is either sprayed on to, or injected into, the soil in a fairly concentrated solution, while the calcium may either be derived from compounds containing it that are already present in the soil, e.g. chalk, or it may also be added in aqueous solution. Calcium chloride is often used to provide soluble calcium salts to react with the sodium silicate. The process can only be used in areas where chalky soils or low-grade limestone aggregate are abundant, thus avoiding the necessity of adding a second solution. The silicate solution employed should be rich in silica, having a ratio of alkali to silica of about 1:3·5, and a specific gravity of from 1·32 to 1·36. At higher concentrations the silicated limestone tends to stick to the roller and to dry out too rapidly. The proportions of binder employed are usually in the range 1 per cent to 10 per cent by weight of the aggregate; this may include stones up to 2 in. in size and certain proportions of limestone filler.

10·18 The material produced in this process is not unlike " lean-mix " rolled concrete, and cubes made at the Road Research Laboratory had compressive strengths as high as 1,000 lb./sq. in. after curing for 28 days.

10·19 Silicated limestone has been widely used in European roads and a review of its practical application has been given by Preslicka[5]. During the war an experimental road section was laid in this country by the Ministry of Transport using a crusher-run soft limestone (" Kentish Rag "). It is considered unlikely, however, that the process is suitable economically for road-building purposes in this country, except in special cases, as the binder is relatively expensive. The principal use appears to be in the injection and grouting techniques employed to increase the stability of excavations and tunnel walls in cohesionless soils. This aspect has been described by Harding and Glossop[6] [7].

(2) Materials which Stabilize the Moisture Content of Soil

10·20 Clay soils become unstable if their moisture content rises to an excessive value; granular soils, on the other hand, lack stability when they are too dry. It is thus desirable to be able to stabilize the moisture content of soils, and materials employed for this purpose can be divided into two groups:—

(a) Waterproofing materials, e.g. bituminous or resinous materials.

(b) Hygroscopic materials, e.g. calcium chloride.

10·21 The properties of bituminous and resinous materials as soil waterproofing agents are discussed in Chapters 13 and 14 respectively. While the use of bituminous materials for this purpose is generally accepted and applicable under field conditions, stabilization with resinous materials is in the development stage, and the available information is derived from laboratory and small-scale field trials.

10·22 Hygroscopic materials can be employed for retaining moisture in the soil. Thus calcium and sodium chlorides, molasses and waste products from the paper industry have been used to some extent, although of these calcium chloride is probably the best known and most widely used, particularly in the U.S.A.[8] and Canada[9].

10·23 The hygroscopic material may be incorporated with the soil base either as a solid or a liquid, with suitable mixing plant, or by the mix-in-place method. Alternatively, it may be added with any water required to achieve the optimum moisture content for compaction, or it may be sprayed on to the compacted base and allowed to soak in.

10·24 The treated soil is similar in its properties to the natural soil; it is not waterproof, owing to the hygroscopic nature of the additive, but the presence of sodium or calcium chloride imparts resistance to the disintegrating action of frost, presumably by reducing the freezing point of the soil water. A disadvantage is that, as all the materials concerned are water-soluble, they tend to be washed out of the soil by rain, and therefore need replacement. Owing to climatic conditions, such chemicals are of little use for stabilizing earth roads in the British Isles, but there are many dry areas in the Colonies and Dominions where they might improve the wearing properties of unsurfaced soil bases.

10·25 The moisture content of soil in road subgrades can also be stabilized by surrounding it with an impermeable membrane made with bituminous material, either unreinforced or strengthened with matting or hessian. A section through a typical membrane-stabilized soil road is shown in Fig. 17·9. This is similar to the design employed in an experimental road laid at the Road Research Laboratory in 1944 (Plate 10·4).

10·26 In the construction of a road of this type, the subgrade is first levelled and compacted, any ditches or drains being excavated simultaneously. A membrane is then laid over the subgrade to prevent any upward movement of moisture, and a layer of natural soil is placed on it and compacted at optimum moisture content. A second membrane is then placed over the compacted and shaped base to prevent the entrance of surface water. This membrane is joined to the first at the haunch, the joint being sealed with a bituminous binder, making a watertight enclosure for the base.

10·27 The road may then be surface-dressed or covered with a bituminous carpet according to traffic requirements. Variations of this method of construction include the use of only the lower membrane to prevent rise of moisture from the foundation, relying on a soil-bitumen mix surfacing for protection against surface water (Morum(10)). Alternatively, when a limited life is required from the road only a surface membrane need be employed with a light surface dressing.

10·28 During the war membrane construction was used on a large scale for the rapid construction of military roads and airfields. For these purposes bituminized jute hessian cloth was employed (prefabricated bituminous surfacing or P.B.S.). This consists of strips of hessian impregnated with soft bitumen and subsequently coated on each side with a harder bitumen containing mineral filler.

10·29 The development and use of P.B.S. for military purposes under western European conditions has been described by Grigson and Lee(11), while Colquhoun and Brock(12) have described a similar material used in India and Burma. A detailed description of the construction of a military road with P.B.S. in Burma has also been given by Colquhoun(13).

Compaction of Bases

10·30 Whatever the type of base material, adequate compaction is essential for ensuring high stability. For stabilized soil bases a small increase in compaction may be equivalent to a considerable increase in the percentage of stabilizer so far as the stability and durability of the base is concerned.

10·31 The necessity for good compaction is further emphasized by the fact that many of the admixtures used in soil stabilization tend to reduce the maximum dry density obtainable with the natural soil under a given set of compaction conditions. In the case of materials which act as binding agents the bonding which increases the stability by raising the cohesion of a soil may also oppose the compaction forces tending to pack the particles together. With certain materials the loss in dry density resulting from the addition of a binding material may more than counteract the gain in cohesion.

Thickness of Bases

10·32 The thickness of base required for an earth road depends on the stability of the subgrade and on the traffic using the road. The estimation of the required thickness is dealt with in Chapter 20. It is considered that for the present purpose empirical methods based on a considerable amount of experience are most suitable. Of these, the method described by the State Highway Department of California[14] is of value.

SURFACINGS

Bituminous Surfacings

10·33 The characteristics of the surfacing will be determined largely by the intensity of the traffic and also by the prevailing climatic conditions rather than by the maximum wheel loads anticipated. Where the traffic is of the order of 50 to 500 vehicles per day a single or double surface dressing is usually adequate, while for more heavily trafficked roads a thin bituminous carpet is desirable. (See Fig. 10·2).
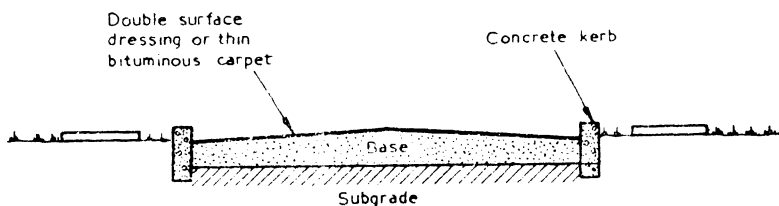
FIG. 10·2 SECTION OF TYPICAL SOIL ROAD FOR URBAN USE e.g. on housing estates

10·34 When a surface dressing is employed without kerbs it is desirable, if practicable, to extend the dressing to cover the haunches to a width of 1 to 2 ft on either side. (See Fig. 10·3.) This gives additional protection to the base by hindering ingress of water from the sides.
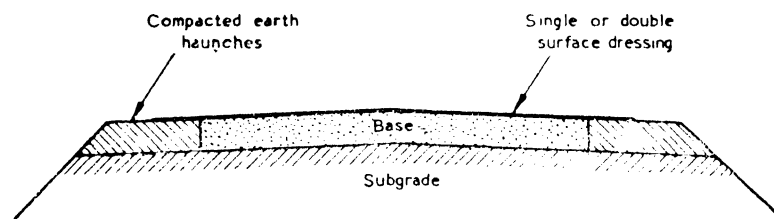
FIG. 10·3 SECTION OF TYPICAL SOIL ROAD FOR USE AS A RURAL OR SECONDARY ROAD

10·35 The bituminous binder employed is of particular importance, since a firm bond is required not only to the aggregate, but also to the underlying base. This is usually achieved by the initial application of a low viscosity oil or cut-

back bitumen as a priming agent, which penetrates the upper surface of the base to a depth of $\frac{1}{4}$ in. or $\frac{1}{2}$ in. This is followed by the application of a binder of higher viscosity which holds the aggregate. The priming process is of particular importance, since the whole surfacing may peel off or be abraded away by traffic if the bond to the base is inadequate. It has been found that such a penetration coat should preferably be applied to a slightly damp rather than to a completely dry base although the soil pores should not be saturated. It may be desirable to allow a base that has been compacted at optimum moisture content to dry slightly prior to spraying.

10·36 The actual composition of the binders employed in surfacing will depend on the type of soil from which the base is constructed, and on the climatic conditions during construction. Typical binders and rates of application for western European and tropical conditions are shown in Table 10·2. This information should be regarded only as a guide however, and the most suitable procedure will in most cases have to be determined by the circumstances.

10·37 Bitumen emulsions are in general unsuitable for priming soil bases owing to their tendency to break as soon as they are applied to the soil. However, in a small-scale experiment in northern Nigeria, it was found that a bitumen emulsion could be made to penetrate quite well into a sand-clay base when it had been watered down to a concentration of about 12 per cent bitumen; higher concentrations failed to give any penetration.

10·38 The aggregate used in the surfacing should preferably be from a hard, clean igneous rock of $\frac{3}{8}$-in. maximum size, but washed gravel may also be used. In order to adhere to the binder, the aggregate should be dry when applied. The rate of spread will vary with the characteristics of the aggregate, but a typical value is 100 to 120 sq. yd/ton.

10·39 More detailed recommendations regarding surface dressing with tar have been given in Road Note No. 1, issued by the Road Research Laboratory in co-operation with the Ministry of Transport[15].

10·40 In cases where local aggregate is not available it has been found possible to oil the surface of the earth road with residual fuel oil, which is then blinded with sand. This tends to give only a sealing effect, however, and it has been stated that it can give a slippery surface when wet[10].

Thin Concrete Surfacings

10·41 Data on the use of thin concrete slabs are gradually accumulating from roads constructed in this country and abroad, in which the slab thickness has been reduced, in some cases to as low as 3 in. It appears that thicknesses of 4 in. and above may give quite satisfactory results for light-traffic roads, provided that the slabs are laid on a thoroughly stable base, e.g. an old water-bound macadam road or a well compacted stabilized soil base. Slabs 3 in. thick have also given good service for some time, when reinforcement is used. Thinner slabs tend to crack at the corners, and it is possible therefore, that 4-in. thick concrete slabs, reinforced at the corners, and laid on stabilized bases may become more common on light-traffic roads as more experience is gained.

TABLE 10·2

TYPICAL BINDERS AND RATES OF APPLICATION FOR SURFACE DRESSINGS FOR EARTH ROADS

Climatic conditions	Treatment	Application	Binder	Rate of spread (gal./sq.yd)
Western European	Prime coat	Cold	RC.1. or MC.1. bitumen Road tar No. 4	0·25-0·33
	Prime coat	Hot	RC.3. or RC.4. bitumen Road tar No. 5	0·25-0·33
	Seal coat	Hot	200-300 pen. bitumen	0·2-0·25
Tropical	Prime coat	Cold	Fuel oil	0·3
	Prime coat	Hot	MC.1. bitumen	0·3
	Seal coat	Hot	MC.4. or MC.5. bitumen	0·2-0·25

Other Surfacings

10·42 A surface course can be constructed from the soil itself by mixing suitable proportions of sand and clay. These proportions must be adjusted so that a well knit, hard and relatively impermeable surface is obtained after compaction. The composition of the soil mixture is fairly critical and specifications suitable for conditions in the U.S.A. have been drawn up by the A.S.T.M.[16]. Similar requirements are specified in a number of other countries, although these are in the main the same as the ones employed for base course work. Soil surface courses are discussed in greater detail in Chapter 11 which also gives some proposed specifications applicable to this country.

10·43 A disadvantage of nearly all soil surfacings is that they are subject to erosion, corrugation and dusting. It has been stated that in Sierra Leone water erosion has caused ruts 12 in. deep in some laterite roads in cuttings. Where corrugation occurs the surface material forms in ridges running transversely to the road at about 2-ft centres and giving an extremely rough riding surface at moderate speeds. It is understood that the effect can be reduced in fairly moist conditions such as those existing in Scandinavia, by using soil of the correct composition. In hotter climates, however, corrugation occurs even on soil roads having a good particle-size distribution and this is believed to be due to the loss of fines as dust clouds, resulting in reduced cohesion. Apart from the corrugation, the raising of dust clouds on dry earth roads is itself a severe nuisance, particularly if a number of vehicles are using a section of the road at the same time.

10·44 The abatement of dusting has been a problem from the earliest days of automobile traffic, and Hubbard[17] listed in 1910 a large number of materials available for dust treatment of earth roads and waterbound macadam. Since then, many road experiments have been made in various countries with different palliatives including calcium chloride in the U.S.A.[18], sulphite lye in Sweden[19] and molasses in India[20]. A summary of existing information has been given by Millard[21] who classified the available dust-palliatives into the following groups:—

(1) Fresh water and sea water.
(2) Deliquescent chemicals, e.g. sodium and calcium chloride.
(3) Bituminous materials.
(4) Organic non-bituminous binders, e.g. sulphite lye.

10·45 It is not known what effect these dust-laying agents have on corrugation but it is felt that, by maintaining the cohesion of the soil fines under dry conditions, they should help to maintain the surface of a soil road intact.

Surface Drainage

10·46 In order that the surfacing can shed water adequately, it should be given a suitable camber or crossfall, or preferably the base should be shaped to the correct section prior to compaction. It is undesirable for water to remain ponded near the kerbs, however, as the action of traffic in such areas may cause disintegration of the thin surfacings that are sometimes used on earth roads, and penetration of water into the base may ensue. Consequently, when kerbs are used, as for example on housing estate roads, gully inlets should

be installed at suitable intervals. Similarly, haunches or berms should not normally be raised above the level of the edge of the surfacing but, where this is unavoidable, grips should be cut through them leading to ditches.

10·47 In tropical areas the runoff from the sealed surface of an earth road may be considerable in periods of heavy rainfall; if uncontrolled, this can lead to severe erosion, not only of the road and the earth haunches but also of the surrounding agricultural land. Suitable remedial measures may therefore be necessary to control the rapid movement of such storm-water, e.g. by grassing the haunches.

SUMMARY

10·48 In many parts of the world earth roads are built with soil or low-grade aggregate, either for reasons of economy and supply, or as stages in the evolution of a high-grade metalled road. They usually have a structure consisting of the subgrade, base and surfacing in the same way as those constructed with higher-grade materials.

10·49 Considerable care is required in construction with low-grade materials, and in particular, special attention must be given to the compaction and drainage of the subgrade.

10·50 The base is the most important component of an earth road, and it should preferably be composed of naturally stable materials such as well graded gravel-sand-clay or sand-clay mixtures, or aggregate that can be crushed to a dense mass under the roller. The stability of the base depends on its thickness, which must be related to the stability of the subgrade and to traffic conditions.

10·51 Admixtures may be incorporated with the base materials to maintain or increase its performance (" soil stabilization "). These can be classified into two groups, viz: those in which the admixture has a binding action and those in which it stabilizes the moisture content of the soil.

10·52 Soil or low-grade aggregate roads are sometimes provided with a wearing course composed of either soil, bituminous material or concrete. When a soil surfacing is used corrugation and dusting are liable to occur and preventive measures in the form of different kinds of surface treatment may be required.

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CHAPTER 11

MECHANICAL STABILIZATION

INTRODUCTION

11·1 When a granular structure, such as a road base or surfacing, has the property of resistance to lateral displacement under load, it is said to be mechanically stable. A good mechanically stable base or surfacing usually consists of a mixture of coarse aggregate (gravel, crushed rock, slag, etc.), fine aggregate (natural or crushed stone, sand, etc.), silt and clay, correctly proportioned and fully compacted. The use of correctly proportioned materials is of particular importance in the construction of low-cost roads and, in fact, the principle of controlled grading is regarded as a method of construction in itself, often termed " mechanical stabilization." Stabilization in this connexion, however, implies increasing the stability of a material under a given set of moisture conditions, rather than maintaining the load-bearing capacity under varying moisture conditions.

APPLICATIONS

11·2 The principle of grading soils may be applied to the improvement of subgrade soils of low bearing capacity, by adding to them materials having particle sizes that are lacking, e.g. sand can be added to clay subgrades and vice versa. It is not usually necessary in such cases to secure a carefully defined size-distribution in the material, but merely to improve the existing distribution by remedying its most obvious deficiencies.

11·3 The main application of the control of the grading of soils and low-grade aggregates is to the construction of bases and sub-bases. The principle of reducing the voids to a minimum has long been recognised in the preparation of all kinds of macadam bases with normal aggregates, and the use of carefully proportioned soil or low-grade aggregate can simply be regarded as an extension of the range of materials that can be employed.

11·4 Low-grade materials have also been extensively employed as surfacings in countries where great lengths of country roads are required. Such surfacings are a big advance on the gravel or earth roads that they have usually replaced, because they are able to carry heavier traffic and are usable over a longer period during the year. Plate 11·1 shows two views of a gravel-sand-clay road in Sweden, where roads of this type have been used extensively and developed to a high degree. The good performance of the Swedish roads is believed to be due to the close particle-size limits employed, which are much narrower than those permitted in other countries.

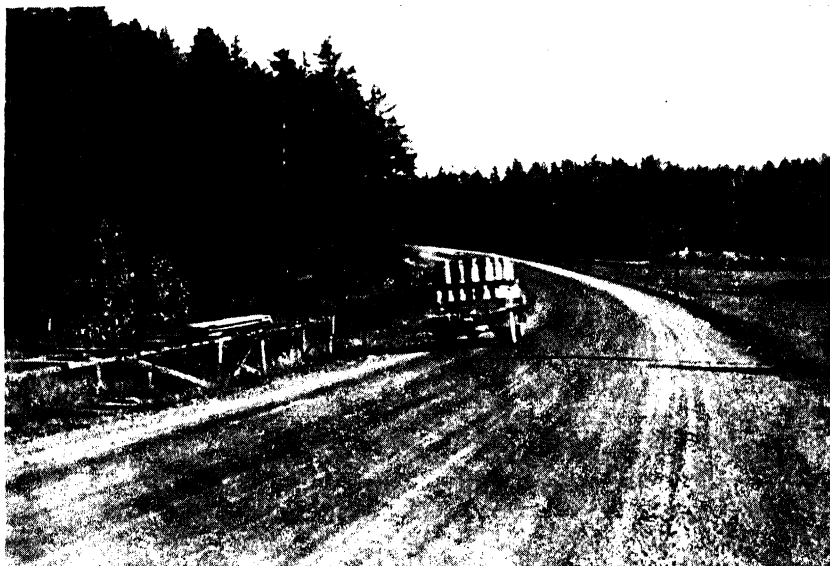
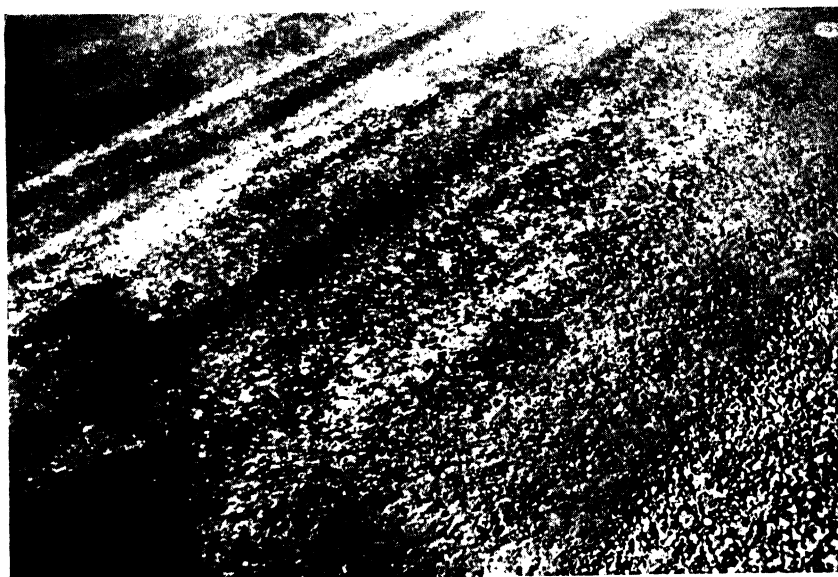
11·5 Many of the hoggins of S.E. England are naturally stable, and have been used as a foundation material in conventional road construction. Soil bases have not, however, been used extensively in this country, although one or two small jobs have been reported. In the U.S.A., Scandinavia, the British

(A) MECHANICALLY STABILIZED ROAD IN SWEDEN
(Surface treated with salt)

(B) CLOSE-UP VIEW OF SURFACE OF ROAD IN (A) ABOVE

PLATE 11·1

dominions and colonies, and in many other countries there is a high proportion of gravel and earth roads, e.g. 60 per cent in the U.S.A., 90 per cent in Sweden, 65 per cent in Poland.

PROPERTIES OF GRANULAR MATERIALS

11·6 The principal properties affecting the stability of compacted granular materials are the internal friction and the cohesion and, to a lesser extent, the compressibility and liability to swelling and shrinkage. These properties have been discussed in greater detail elsewhere in this book, principally in Chapter 2. It is sufficient to note here that internal friction is chiefly characteristic of the coarser soil particles such as gravels, sands and silts, while cohesion, compressibility, swelling and shrinkage are mainly associated with the clay fraction.

FACTORS AFFECTING MECHANICAL STABILITY

Mechanical Strength of the Aggregate

11·7 It is probable that in a well proportioned and thoroughly compacted mechanically stabilized road a low mechanical strength is permissible in the aggregate, and it is thought that most aggregates with crushing strengths above about 12,000 lb./sq.in. (aggregate crushing values below 40) [1] will be sufficiently strong for most purposes.

11·8 It has been suggested that weak aggregate is to be preferred for mechanical stabilization, because it will break down under compaction to give a size distribution more closely approaching that required for maximum dry density (see section on particle-size distribution). Although this may be the reason why very weak aggregates sometimes give better results than are expected, a preference for weaker aggregates can hardly be recommended on these grounds. Wherever possible the aggregate should be correctly proportioned before laying and should have sufficient mechanical strength to retain approximately the same size distribution during compaction and subsequent use by traffic.

Mineral Composition of the Materials

11·9 Experience suggests that almost any material that is résistant to weathering will be suitable for use as aggregate in a mechanically stabilized road; all kinds of natural rock, gravel and sand, and artificial materials (such as slag, burnt shale, etc.) have been used with success. Low-grade materials such as laterite, coral and limerock are also known to give dense, mechanically stable roads.

11·10 The mineral composition of the fines passing the No. 200 B.S. sieve, i.e. the silt and clay, is known to affect the cohesion of a soil; in particular, the presence of certain micas has been found undesirable [2]. It is usual to specify the characteristics of the fines in terms of soil plasticity tests.

Particle-size Distribution (grading)

11·11 To achieve mechanical stability it is necessary to have a well proportioned coarse material containing some clay binder. For the coarse material it is usually assumed that the particle-size distribution giving the greatest dry

density has the greatest internal friction, although this has not been proved experimentally. The work of Fuller[3], later confirmed by Rothfuchs[4] and others, showed that a granular mass has a relatively high dry density when its particle-size distribution follows a certain law, which for practical purposes may be written thus:—

$$\text{Percentage passing any sieve} = 100\sqrt{\frac{\text{The aperture size of that sieve}}{\text{The size of the largest particle}}}$$

11·12 Experience shows, however, that to obtain sufficient cohesion it is usually necessary to have a greater proportion of material passing No. 200 B.S. sieve than is given by this formula, especially for surfacings. When an adjustment has been made for this purpose, it is possible to draw up a series of limits of size distribution for aggregates of different maximum particle size, which probably represent the most highly stable compositions that can be obtained, and which can be used as a guide to the type of aggregate mixture that is required in practice. Just how far the size distribution of an aggregate can deviate from the " Fuller " curves without serious loss of mechanical stability is not known, but experience suggests that a wide range of limits may be suitable.

11·13 The limits of size distribution recommended in a number of American, Canadian, Swedish and Norwegian papers and specifications have been studied, and in nearly all cases they have been found to approximate to the Fuller curves. The tolerances permitted vary considerably, but the general impression gained from this study is that present knowledge does not justify the application of narrow limits, and that practical limitations preclude it.

11·14 In the U.S.A., specification limits (Table 11·1)[5] have been based on materials that have been found to give satisfactory results in practice. It is admitted that many aggregates which fail, for one reason or another, to comply with the requirements given in Table 11·1 have been found satisfactory in practice; all that is claimed for the specification is that materials which comply with it are more likely to be satisfactory for mechanical stabilization than those which do not.

11·15 The limits of size distribution given in this American specification are expressed in the cumulative form, i.e. in the form of the total percentage passing each sieve. This method of specifying particle-size limits suffers from the disadvantage that if reasonably wide tolerances are allowed on the percentage passing each sieve, it is possible for some fractions to be completely missing. This failing is particularly objectionable in the specification of aggregates for mechanical stabilization, in which it is highly desirable that a fair proportion of each particle-size fraction should be present. In Table 11·1, the type B aggregate 2-in. base can have 50 per cent passing both the ¾-in. and the No. 7 B.S. sieves, so that the fraction between these sieves can be absent, which was certainly not intended by those who wrote the specification. Hogentogler [6] mentions a case of failure of a base that was attributed entirely to a deficiency in but one of the sand fractions.

11·16 The easiest way to surmount this difficulty is to specify, in addition to the percentage passing each sieve, that not less than a certain amount must be retained between each pair of successive sieves. This has been done in Table

TABLE 11·1

AMERICAN TENTATIVE LIMITS OF PARTICLE-SIZE DISTRIBUTION FOR BASES AND SURFACINGS

(A.S.T.M.: D 556-40T and A.S.T.M.: D 557-40T)

B.S.* sieve size	Percentage passing				
	TYPE A[x] (Sand-clay) Base or surfacing	TYPE B (Gravel-sand-clay)			TYPE C Base or surfacing
		Base		Surfacing	
		2-in. max. size	1-in. max. size		
2 in.	—	100	—	—	—
1½ in.	—	70 — 100	—	—	—
1 in.	—	55 — 85	100	100	—
¾ in.	—	50 — 80	70 — 100	85 — 100	100
⅜ in.	—	40 — 70	50 — 80	65 — 100	—
3/16 in.	—	30 — 60	35 — 65	55 — 85	70 — 100
No. 7	100[x]	20 — 50	25 — 50	40 — 70	35 — 80
No. 18	55 — 90	—	—	—	—
No. 36†	35 — 70	10 — 30	15 — 30	25 — 45	25 — 50
No. 200‡	8 — 25	5 — 15	5 — 15	10 — 25	8 — 25

*The nearest equivalent commonly used B.S. sieves are given here.

†The fraction passing the No. 36 sieve shall have the following characteristics:—

For bases Liquid limit not exceeding 25 per cent.
Plasticity index not exceeding 6 per cent for types A and B, 3 per cent for type C.

For surfacings Liquid limit not exceeding 35 per cent.
Plasticity index between 4 and 9 per cent.

‡The percentage passing the No. 200 sieve shall be not more than one-half of that passing the No. 36 sieve for bases, and two-thirds for surfacings.

[x]Type A mixtures may have up to 35 per cent retained on the No. 7 sieve; the limits shown are those of the material passing the No. 7 sieve.

11·2, which gives tentative suggestions for aggregates for mechanical stabilization based on the study of the Fuller curves and published specifications mentioned above. Table 11·2 is intended to provide guidance in the selection or mixing of aggregates for mechanical stabilization, and is not intended to be applied as a rigid specification. The main requirement is that mixtures shall contain a fair proportion of all the particle-size fractions from the largest to the smallest, with sufficient of the smallest to provide cohesion, especially in the wearing surface. Table 11·2 gives examples of such mixtures in various maximum sizes; other similarly proportioned mixtures would probably be equally satisfactory. The limits of size distribution from Table 11·2 are plotted in Figs. 11·1 to 11·6, the nearest Fuller curve of maximum dry density being added to each diagram. The requirements of Table 11·2 mean that the curve of size distribution of an aggregate or mixture of aggregates for mechanical stabilization should lie within the shaded area of one of these diagrams, and should be approximately parallel to the limiting curves.

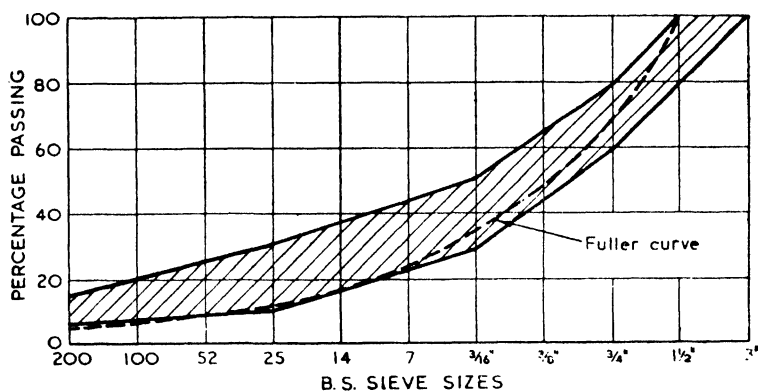



FIG. 11·1 SUGGESTED PARTICLE-SIZE LIMITS FOR BASES
3-in. maximum aggregate size

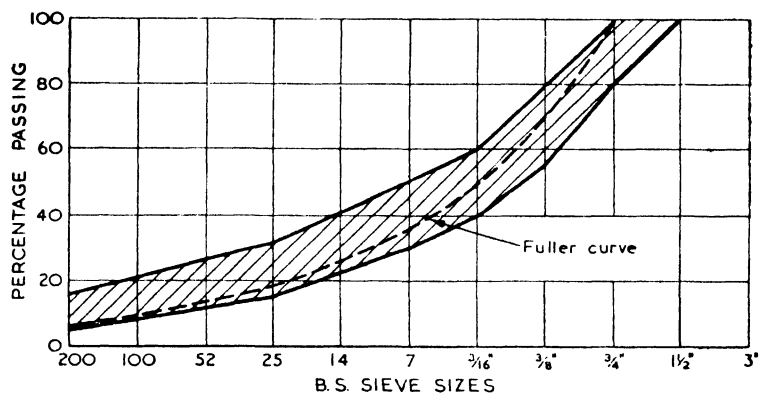



FIG. 11·2 SUGGESTED PARTICLE-SIZE LIMITS FOR BASES
1½-in. maximum aggregate size

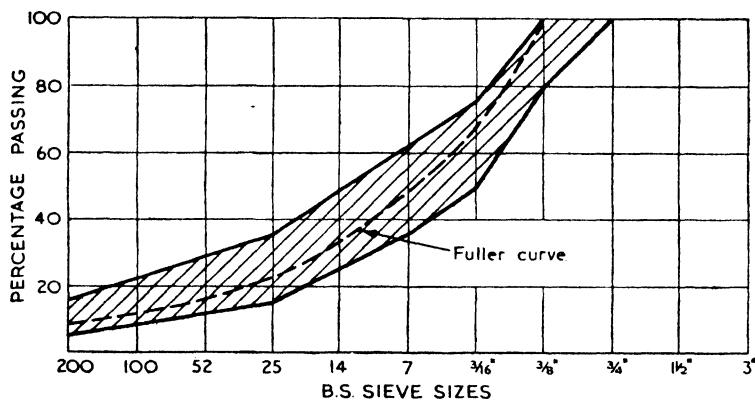



FIG. 11·3 SUGGESTED PARTICLE-SIZE LIMITS FOR BASES
¾-in. maximum aggregate size

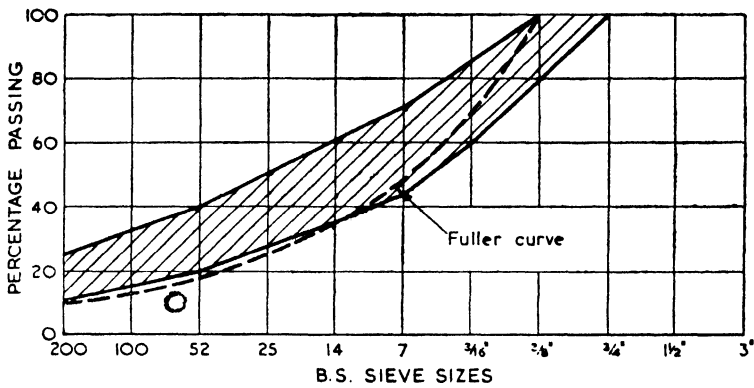



FIG. 11·4 SUGGESTED PARTICLE-SIZE LIMITS FOR SURFACINGS
$\frac{3}{4}$-in. maximum aggregate size

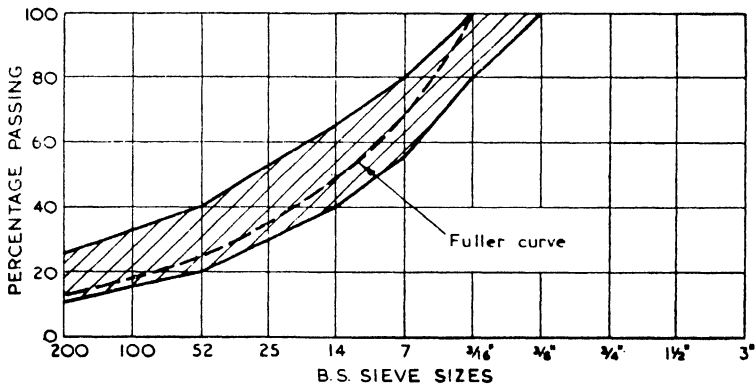



FIG. 11·5 SUGGESTED PARTICLE-SIZE LIMITS FOR BASES OR SURFACINGS
$\frac{3}{8}$-in. maximum aggregate size

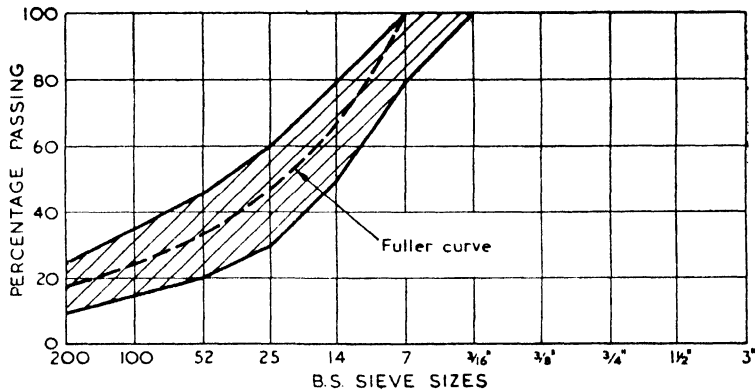



FIG. 11·6 SUGGESTED PARTICLE-SIZE LIMITS FOR BASES OR SURFACINGS
$\frac{3}{16}$-in. maximum aggregate size

TABLE 11·2

PROPOSED LIMITS OF PARTICLE-SIZE DISTRIBUTION FOR BASES AND SURFACINGS, WITH TOLERANCES

B.S. sieve size	Percentage passing					
	Base			Surfacing	Base or surfacing	
	Nominal maximum size			Nom. max. size	Nominal maximum size	
	3-in.	1½-in.	¾-in.	¾-in.	3/8-in.	3/16-in.
3 in.	100	—	—	—	—	—
1½ in.	80 — 100	100	—	—	—	—
¾ in.	60 — 80	80 — 100	100	100	—	—
3/8 in.	45 — 65	55 — 80	80 — 100	80 — 100	100	—
3/16 in.	30 — 50	40 — 60	50 — 75	60 — 85	80 — 100	100
No. 7	—	30 — 50	35 — 60	45 — 70	50 — 80	80 — 100
No. 14	—	—	—	35 — 60	40 — 65	50 — 80
No. 25	10 — 30	15 — 30	15 — 35	—	—	30 — 60
No. 52	—	—	—	20 — 40	20 — 40	20 — 45
No. 200	5 — 15	5 — 15	5 — 15	10 — 25	10 — 25	10 — 25

Notes

1. Not less than 10 per cent should be retained between each pair of successive sieves specified for use, excepting the largest pair.
2. The two smaller sized materials (3/8- and 3/16-in.) may have up to 35 per cent of stones not larger than 1½-in., provided that the material passing the 3/16-in. sieve is within the limits specified.
3. The material passing the No. 36 sieve shall have the following characteristics (B.S. 1377: 1948);—

   For bases Liquid limit not exceeding 25 per cent.
   Plasticity index not exceeding 6 per cent.

   For surfacings Liquid limit not exceeding 35 per cent.
   Plasticity index between 4 and 9 per cent.

The Properties of the Soil Mortar

11·17 These are determined by the B.S. plasticity tests, i.e. the liquid and plastic limit tests, which are carried out on the fraction of the soil passing the No. 36 B.S. sieve. The results obtained are dependent on the amount and type of clay present. The experimental details of these tests are described in Chapter 3.

11·18 It has been found in practice[6] that the plasticity index should be limited to a maximum of 6 per cent for bases and should be between 4 per cent and 9 per cent for surfacings. The liquid limit should not exceed 25 per cent for base material and 35 per cent for surfacings. The higher liquid limits and plasticity indices are desirable in the surfacing in order to provide greater cohesion and to help offset the moisture lost by evaporation. They are, however, quite undesirable in the base, and should not be permitted in any temporary surfacing that is intended ultimately to receive some other, more permanent, surfacing.

Compaction

11·19 Adequate compaction is very important when constructing roads with soils or low-grade aggregate mixtures. Dry densities of over 140 lb./cu.ft—approaching 90 per cent of the bulk density of rock—can be obtained with well proportioned granular materials. The plant required for compacting such materials and the laboratory and control tests required are described in more detail in Chapter 9.

11·20 Experience has shown that good compaction of a base can be obtained if it is used by traffic for some months before a surfacing is applied. Calcium chloride has also been found to facilitate compaction and is useful in preserving the stability of a base when temporarily used as a surfacing.

PROPORTIONING THE MATERIALS

11·21 The particle-size distribution of some natural materials, such as many of the "hoggins" and "wall-ballasts" of S.E. England, are within the limits required for mechanical stability and the materials contain sufficient clay to act as a binder. More often, however, natural materials are deficient in one or more of the particle-size fractions required, and a mechanically stable material can be prepared only by mixing two or more of them in the correct proportions.

11·22 Numerous methods have been described for determining the proportions in which materials of known sizes must be mixed to produce a specified size distribution. The method described by Rothfuchs[(7)] has been found most useful, as it is reasonably quick and simple and can be applied to mixtures of any number of components. It consists essentially of the following stages:—

(1) The cumulative curve of the required aggregate particle-size distribution is plotted, using the usual linear ordinates for the percentage passing but choosing a scale of sieve size such that the particle-size distribution plots as a straight line. This is readily done by drawing an inclined straight line and marking on it the sizes corresponding to the various percentages passing.

(2) The particle-size distribution curves of the aggregates to be mixed are plotted on this scale. It will generally be found that they are not straight lines.

(3) With the aid of a transparent straight edge, the straight lines that most nearly approximate to the particle-size distribution curves of the single aggregates are drawn. This is done by selecting for each curve a straight line such that the areas enclosed between it and the curve are a minimum and are balanced about the straight line.

(4) The opposite ends of these straight lines are joined together, and the proportions for mixing can be read off from the points where these joining lines cross the straight line representing the required mixture.

11·23 The actual procedure will be apparent from the following example:—
Columns 4, 5, and 6 of Table 11·3 give the particle-size distribution of a crusher-run stone, a sand and a silty clay that it is required to mix to produce a mechanically stable surface within A.S.T.M. limits. The average of these limits is given in Table 11·3, column 3, and is the required size distribution.

(1) The required size distribution is represented by the diagonal 00′ of a rectangle (Fig. 11·7). The vertical ordinates of the rectangle are graduated for percentages from 0 to 100 on a linear scale. The horizontal scale for sieve aperture size is graduated by drawing for each sieve size a vertical line that cuts the diagonal at a point where the ordinate equals the percentage passing that sieve, i.e. 100 per cent for 1 in., 92 per cent for $\frac{3}{4}$ in., 82 per cent for $\frac{3}{8}$ in. and so on.

(2) The size distribution of the aggregates to be mixed (Table 11·3, columns 4, 5 and 6) are plotted on this scale of sieve size (Fig. 11·7), giving the lines BAO′ (crusher-run), BFE (sand) and OG (silty clay).

(3) The nearest straight lines to these size distributions are drawn with the aid of a transparent straight-edge, by the " minimum balanced areas " method described above. They are the dotted lines CO′, BD and OG (the last being coincident with the actual size distribution).

(4) The opposite ends of these lines are joined, giving the chain lines CD and BG (in this case, the latter coincides with the No. 200 B.S. sieve ordinate). The points where these lines cross the required size-distribution line are marked by the circles L and M. The proportions in which the three aggregates should be mixed are obtained from the differences between the ordinates of these points, and are shown on the right-hand side of Fig. 11·7.

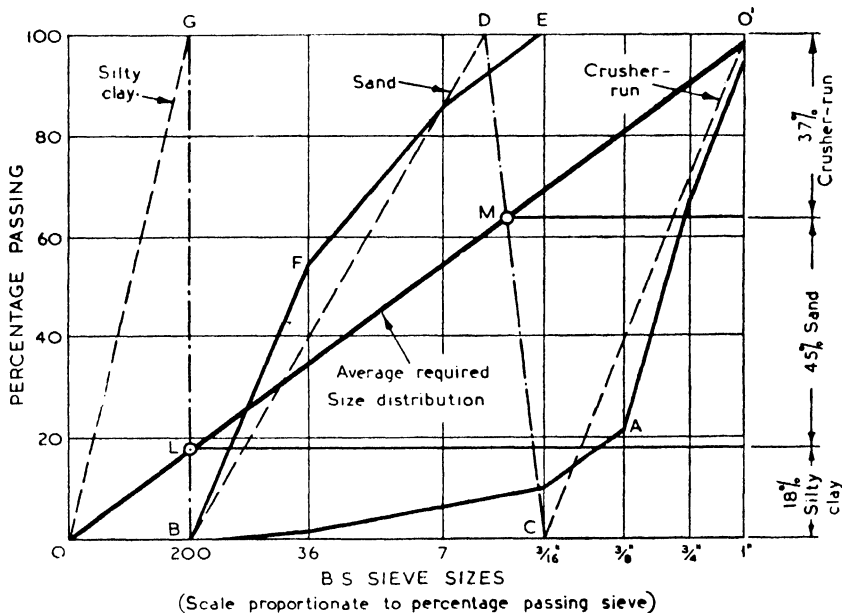



FIG. 11·7 DETERMINATION OF MIXTURE FOR STABILIZED SURFACINGS

11·24 The particle-size distribution that will result from mixing the aggregates in these proportions is given in column 7 of Table 11·3. Although not identical with the required size distribution (column 3), it is within the specified limits (column 2).

TABLE 11·3

EXAMPLE OF ROTHFUCHS' METHOD FOR PROPORTIONING MIXTURES OF AGGREGATE

(See Fig. 11·7)

MIXTURE FOR SURFACINGS

(Table 11·1, type B)

1	2	3	4	5	6	7
B.S. sieve size	Percentage passing					Mixture 37% A 45% B 18% C
	Required size distribution		Aggregates available			
	Limits	Average	(A) Crusher-run	(B) Sand	(C) Silty clay	
1 in.	100	100	95	—	—	98
¾ in.	85 — 100	92	70	—	—	89
⅜ in.	65 — 100	82	21	—	—	71
3/16 in.	55 — 85	70	11	100	—	67
No. 7	40 — 70	55	7	85	—	58
No. 36	25 — 45	35	2	55	—	43
No. 200	10 — 25	18	Trace	Nil	100	18

11·25 In order to meet the plasticity requirements for the fines passing the No. 36 B.S. sieve, it is sometimes necessary to mix different types of soil. Hogentogler[6] gives a method for calculating the proportions for such mixes, but if the percentage passing the No. 200 B.S. sieve is kept well within the specified limits, this operation will rarely be necessary.

LABORATORY TESTING TECHNIQUES

11·26 The laboratory tests required to control the composition of soils and low-grade aggregates are the particle-size analysis and the B.S. plasticity tests, see Chapter 3.

11·27 In addition, it is often informative to carry out a B.S. compaction test (see Chapter 9) on the material or mixture of materials, and subsequently to carry out a California bearing ratio test on a sample compacted to the dry density and moisture content obtained in the compaction test. A description of the procedure for the California bearing ratio test is given in Chapter 19.

CONSTRUCTION

11·28 The methods employed in constructing soil or aggregate roads are similar to those employed in soil stabilization processes, i.e. mix-in-place, travelling or stationary plant methods may be used. These are described in detail in Chapter 15.

11·29 A particularly economical form of construction peculiar to this type of work is the treatment of heavy clay, e.g. black cotton soil with sand, which has been used with some success in Nigeria. Sand is spread on top of, but not mixed with the clay, and blending is achieved by the combined action of traffic and weather until the surface layer of the soil has a good size-distribution.

FIELD CONTROL TESTS

11·30 In the field the quality of the material produced is controlled by determining the composition (by a particle-size analysis), and measuring the moisture content and dry density. Suitable methods are available for determining the particle-size distribution and the moisture content of a soil under field conditions, and the details of these are given in Chapter 3. The procedures for the B.S. plasticity tests, which are used to control the properties of the soil mortar, are the same under both laboratory and field conditions.

11·31 The dry density to which material has been compacted in the field can conveniently be checked by the sand-replacement method, described in Chapter 9. Where very large stones are present it may be necessary to employ a modification of this method, described in B.S. 1377:1948 (Test No. 10B).

MAINTENANCE

11·32 Soil road surfaces require periodic blading, so as to maintain a smooth surface and adequate crown to shed rain water. During dry weather the surface should be kept free from loose aggregate, which should be bladed to the side. If a considerable amount of such loose material accumulates on the side of the road, it can be mixed with a suitable quantity of soil binder and water to stabilize it, and re-spread and compacted on the road surface, any serious ravelling, rutting or pot-holes having previously been made good. During light rain, chatter bumps, pot-holes and other minor irregularities should be removed by light blading from the centre to the sides of the road, the material being bladed back and shaped after the rain has ceased but while still moist. A longer interval will be required before blading back after heavy rain, and in no case should material be bladed back from the sides to the centre of the road whilst rain is still falling. Calcium chloride can be worked into the surface at the rate of about 1 lb./sq.yd once or twice a year after rain, in order to maintain cohesion in dry weather.

SUMMARY

11·33 It is possible to utilize the inherent properties of the materials to provide mechanical stability when constructing roads with soils or low-grade aggregates. The most important of these properties are internal friction and cohesion, the former being characteristic of the coarse material and the latter of the fine. The extent to which a soil or aggregate possesses these properties is determined largely by its particle-size distribution, the main requirement being that it shall contain a fair proportion of all sizes. A higher proportion of the finer sizes is required in the surfacing than in the base, to assist in the retention of moisture necessary for cohesion.

11·34 The types of mixture required are indicated in tabular and graphical form. Some hoggins and ballasts have naturally stable compositions but usually the required size distribution has to be obtained by mixing materials that would be unsuitable in themselves. A method for proportioning such mixtures is given.

11·35 The principle of controlled composition has been used successfully in the improvement of weak subgrades, in the construction of bases to take all types of surfacings, and in the construction of surfacings for country roads. The plant required and methods of construction are similar to those for other types of stabilization.

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CHAPTER 12

STABILIZATION OF SOIL WITH CEMENT

INTRODUCTION

12·1 In the stabilization of soil with cement, between 5 and 15 per cent by weight of cement is added to the soil to produce a material, " soil-cement ", which is stronger and more durable than the untreated soil. Soil-cement was first employed in road construction in the U.S.A. in 1935, and since then has been used on an increasing scale, until by 1949 there were many millions of square yards of this type of construction in many parts of the world. Most of this work has been carried out in the U.S.A. and in the Union of South Africa, about 50 million sq.yd having been constructed in the U.S.A. alone. In 1950 there were over 660,000 sq. yd of soil-cement in this country, half of which had been constructed since 1945. Soil-cement has also been used to a certain extent for building purposes, including the manufacture of building blocks and as foundations for houses.

APPLICATIONS

12·2 The ways in which soil-cement may be used in road construction depend on the soil type, site conditions and the amount of traffic anticipated. Table 12·1 shows the ranges of compressive strengths obtained with soil-cement made with different soils, in relation to the strength of untreated soil and concrete, and also indicates the uses of the different materials. The values of compressive strength quoted are intended as a general guide and not as an aid to design.

12·3 During the war soil-cement was used in this country to construct airfields. Since the war there has been an increasing tendency to use soil-cement in the construction of housing estate roads, and it has also been employed for the sub-bases of major roads. Other uses have been in the construction of builders' roads, storage depot roads, shipyards, car parks, footpaths and foundations for large water-storage tanks. Table 12·2 gives the approximate areas of different types of soil-cement constructed in this country by 1950.

12·4 It is likely that construction using soil-cement will have considerable application in colonial development, where new roads are required at costs lower than those for the more conventional forms of construction. A further application of soil-cement has been developed in the U.S.A. in which a very wet mix known as " plastic soil-cement " is employed as a mortar for the construction of linings for ditches, canals and reservoirs, as well as for small areas inaccessible to large plant.

TABLE 12·1

TYPICAL COMPRESSIVE STRENGTHS OF SOILS, CONCRETE, AND SOIL-CEMENT MIXTURES CONTAINING ABOUT 10 PER CENT OF CEMENT

Compressive strength range (lb./sq. in.)	Material	Suggested use
	Soils	
<10	Clay, peat	Subgrades
10—40	Well compacted sandy clay	Subgrades
40—100	Well compacted gravel-sand-clay mixtures	Subgrades
	Soil-cement made from:	
<50	Clays, organic soils	Should not be used
50—150	Silts, silty clays, very poorly graded sands, slightly organic soils	Sub-base on very poor foundation
100—250	Silty clays, sandy clays, poorly graded sands and gravels	Sub-base on poor foundations; paths or cycle tracks (with surfacing)
250—500	Silty sands, sandy clays, sands and gravels	Base for minor roads in temperate climates (with surfacing); base for main roads or runways
400—1,500 (approx.)	Well graded sand-clay or gravel-sand-clay mixtures and sands or gravels	Base for minor roads in more severe climates (with surfacing); base for medium-class roads in temperate climates (with surfacing); sub-base for main roads and runways
N.B.—Strengths up to 4,000 lb./sq. in. have been recorded.		
	Concrete	
500—2,000	Lean-mix concrete	Base or sub-base for main roads or runways
2,000—5,000	Concrete	Base for main roads or runways

FACTORS AFFECTING THE PROPERTIES OF SOIL-CEMENT

12·5 The chief factors affecting the quality of soil-cement are soil type, cement content, compaction and method of mixing. Of these the soil type is by far the most important factor since, if it is unsuitable, little can be done to make the soil-cement satisfactory.

Soil Type

12·6 This refers primarily to the particle-size distribution and the chemical composition of the soil. The Portland Cement Association of America[1], states that almost any soil which can be pulverized can be stabilized with cement, but it may be uneconomical at the present time to pulverize some soils such as heavy clays. The following limits are based on those suggested by the Highway Research Board of America[2] for soils that can be economically stabilized:—

Particle-size distribution limits:—		B.S. plasticity test limits:—	
Maximum size	3 in.	Liquid limit	$<40\%$
Passing $\frac{3}{16}$-in. B.S. sieve	$>50\%$	Plasticity index	$<18\%$
Passing No. 36 B.S. sieve	$>15\%$		
Passing No. 200 B.S. sieve	$<50\%$		

12·7 Soils that have been successfully stabilized in this country range from materials with clay contents up to 30 per cent, requiring 12-15 per cent of cement, to sand-clay mixtures giving high strengths and durability with cement contents of from 5 to 8 per cent. Quarry dust and certain shales have also been used successfully.

12·8 The soil should be low in organic matter for successful stabilization since this constituent tends to reduce the strength of soil-cement. Although in some cases soils with 3 to 4 per cent of organic matter have been successfully stabilized, about 2 per cent is considered to be the safe upper limit and, with some otherwise suitable soils, as little as 0·5 per cent of organic matter has rendered the soil unsuitable for stabilization. In some cases, e.g. with sandy soils from pine forests, organic soils have been rendered more suitable for processing by the admixture of up to 25 per cent of another soil, or of 0·6 to 1·0 per cent of calcium chloride, while the use of lime for this purpose is also reported.

12·9 Apart from organic matter, the chemical composition of the soil is believed to be of importance only if appreciable quantities of deleterious salts are present. By analogy with concrete technology, it is probable that the sulphates are the most harmful. The harmful effect of these compounds is thought to be due not to a reaction affecting the setting of the cement, but to a subsequent disruption of the soil-cement structure caused by crystallization of highly hydrated salts in the pores. Such effects, which are usually associated with the movement of moisture in the soil-cement or in the surrounding soil, have been observed in the laboratory, but it is not known to what extent they may occur in the field.

Cement content

12·10 Soil-cement has been made successfully with a cement content as low as 4 per cent in many cases, chiefly in South Africa, while in other cases as much as 20 to 25 per cent may be required to give a satisfactory strength, although it is unlikely that the use of such a high proportion of stabilizer would be economical.

12·11 In this country the cement content required is usually determined by measuring the compressive strength of specimens made with different proportions of cement. The moisture content at which these are made and the dry density to which they are compacted are determined in the manner described in a later section of this chapter. When the cement is hydrating satisfactorily in the mixture, an increase in strength is obtained with increasing cement content. Table 12·3 shows the increases in compressive strength obtained by raising the cement content of various soil-cement mixtures, using soils whose particle-size analyses are given in Fig. 12·1. A given increase in the cement content with the more clayey soils, e.g. soil 1, produced a smaller increase in compressive strength than with sandy soils, e.g. soil 4. The effect of the presence of coarse aggregate (soil 5) was to give a relatively low compressive strength for lean mixes, but a relatively high compressive strength for rich mixes.

TABLE 12·2

AREAS OF SOIL-CEMENT CONSTRUCTION IN GREAT BRITAIN (1950)

Type of construction	Area (sq. yd)
Sub-bases for major roads	147,000
Housing estate roads	203,000
Other minor roads	9,000
Airfields	109,000
Military roads	29,000
Car parks, storage depots, shipyards, reservoirs, footpaths, etc. ...	165,000
Total ...	662,000

12·12 In this and some other countries, the cement content is specified as a percentage by weight, but in most parts of the U.S.A. and elsewhere it is specified as a percentage by volume, whilst in Germany it has been specified in kilograms per square metre for a given depth of treatment.

12·13 As already indicated, for most soils the effect of an increase in cement content is an increase in strength, and with it an increase in durability. Soils with which this does not occur are generally unsuitable for use in soil-cement construction. It has been found at the Road Research Laboratory that with some soils the initial rate of increase of strength after mixing and compaction is the same over a fairly wide range of cement contents so that cubes tested within a day or two after preparation might give the same strength with, say,

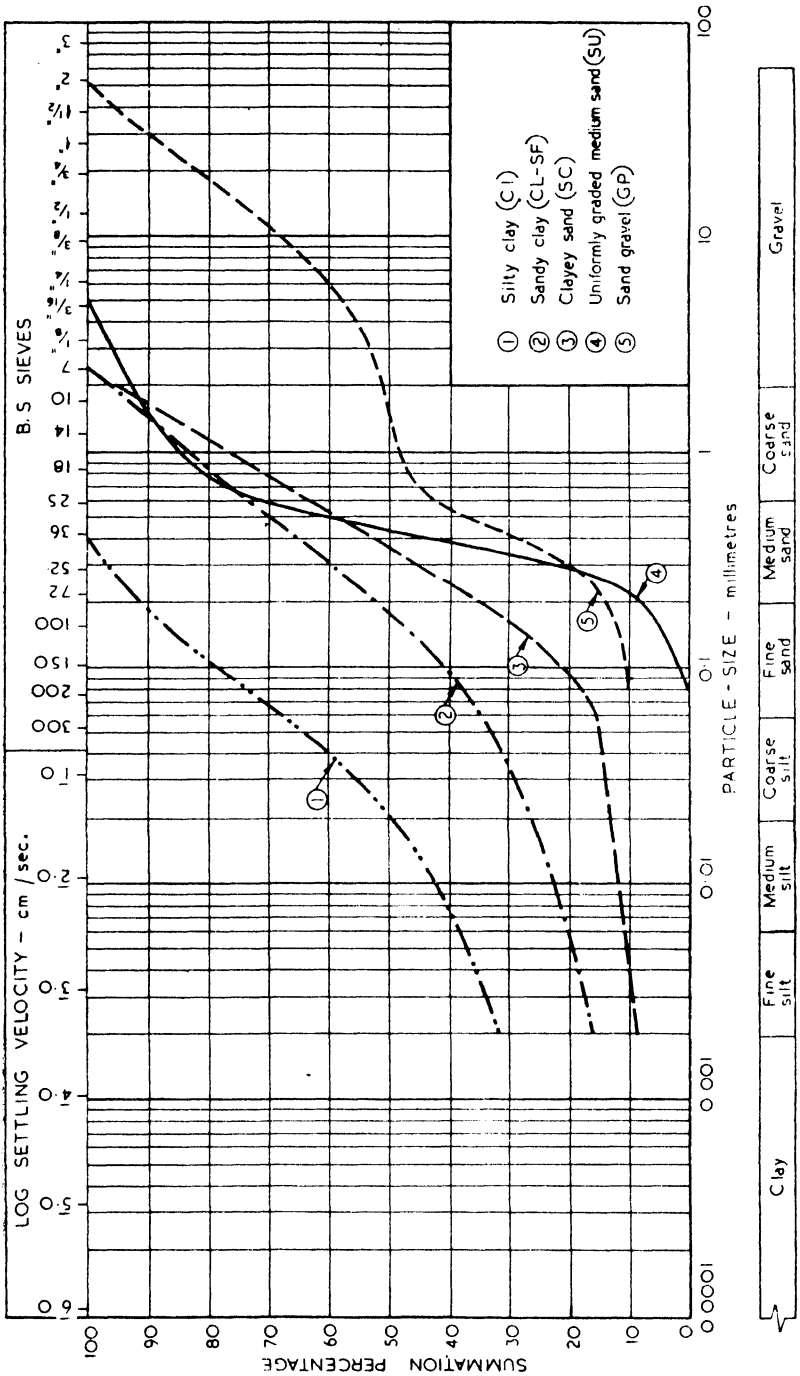



FIG. 12·1 PARTICLE-SIZE DISTRIBUTION OF VARIOUS SOILS USED FOR SOIL-CEMENT

TABLE 12·3

TYPICAL COMPRESSIVE STRENGTHS OF SOIL-CEMENT MIXTURES. SHOWING INCREASE IN STRENGTH WITH INCREASING CEMENT CONTENT

Soil	Cement content (%)	Compressive strength (7 days) (lb./sq. in.)	Dry density (lb./cu. ft)	Moisture content (%)
1. Silty clay CI	7	350	111	16
	10	400	111	
	13	450	111	
2. Sandy clay CL-SF	7	260	117	14
	10	380	118	
	13	530	118	
3. Clayey sand SC	7	240	111	12
	10	280	113	
	13	390	115	
4. Clean sand (uniformly graded) SU	7	210	111	10
	10	410	115	
	13	860	118	
5. Gravel (poorly graded) GP	7	160	124	10
	10	360	125	
	13	560	127	

12 per cent of cement as with 8 per cent. If the cubes are left for seven days or longer, this phenomenon ceases in the majority of cases, and cubes with a higher cement content do give a higher strength. It is not considered generally necessary to wait 28 days before testing.

Moisture content

12·14 The effect of moisture content on the quality of soil-cement largely arises from its influence on the compaction; for good compaction is necessary to bring the materials to the maximum dry density for a given effort. The best moisture content for compaction is governed by the soil type and method of compaction, and the water/cement ratio concept as used in concrete work is therefore of little value in soil-cement stabilization. Work at the Road Research Laboratory has shown that there is a tendency for the strength to increase with moisture content for a given dry density. Unpublished work reported from South Africa shows that, although the highest compressive strength can be obtained with specimens compacted slightly below the optimum, the best results in durability tests are obtained at moisture contents somewhat above the optimum.

12·15 There is no conclusive information as to the best moisture content for mixing soil and cement. Tests have been carried out at the Road Research Laboratory with both field and laboratory mixers[3] on the uniformity with which powdered chalk and cement can be mixed with a sandy clay, and cement with a silt clay. The results indicate that the best mixing is generally obtained either with the soil considerably wetter than at the optimum moisture content for B.S. compaction, or with very dry soil (Fig. 12·2). As the latter condition

is neither attainable in this country nor desirable in practice it is considered that, until further evidence is available, mixing should as far as practicable be carried out at a moisture content at or somewhat above the optimum for the compaction plant available.

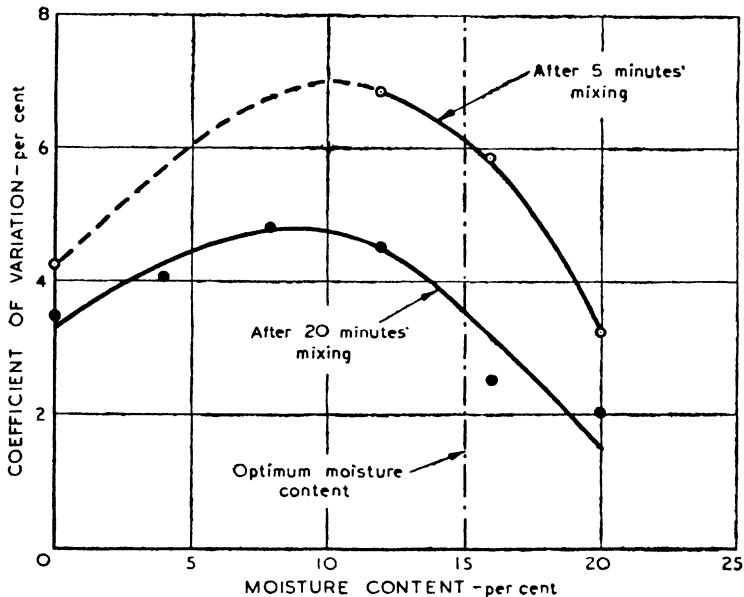



FIG. 12·2 RELATIONSHIP BETWEEN INITIAL MOISTURE CONTENT AND COEFFICIENT OF VARIATION* OF CHALK CONTENT FOR SAMPLES OF SOIL MIXED IN A LABORATORY MIXER

12·16 The moisture required for the hydration of the cement is adequately provided by the moisture necessary for maximum compaction. This should be well distributed throughout the soil, however, so that all the cement is in intimate contact with sufficient moisture. Since cohesive soils are naturally retentive of moisture it is desirable to add an additional quantity of moisture after the soil and cement have been mixed, so that there will be no dry places where the cement cannot hydrate.

12·17 Except in unusually dry summers, excavated soils in the British Isles will usually be found at or above the optimum moisture content, but they will very often dry during processing so that water may have to be added. In such cases the same precautions should be taken to use pure water as would be taken in concrete work, i.e. it should be free from excessive amounts of oils, alkalies, salts or organic matter.

Compaction

12·18 To obtain satisfactory soil-cement adequate compaction is essential; this is shown by the fact that in the laboratory a decrease in dry density of 1 lb./

*See footnote p. 119, Chap. 6, for definition of coefficient of variation.

cu. ft has been found to cause a decrease in compressive strength of between 20 and 40 lb./sq. in. and a greater proportionate loss of durability. A typical compressive strength/dry density curve is given in Fig. 12·3, which relates to a sandy clay mixed with 10 per cent of cement. A 5-per cent decrease in the relative compaction is stated by Stanton, Hveem and Beatty[4] to cause a greater drop in compressive strength than a decrease of 10 or 15 per cent in the amount of cement, e.g. a decrease from 10 per cent of cement to 9 or 8½ per cent.

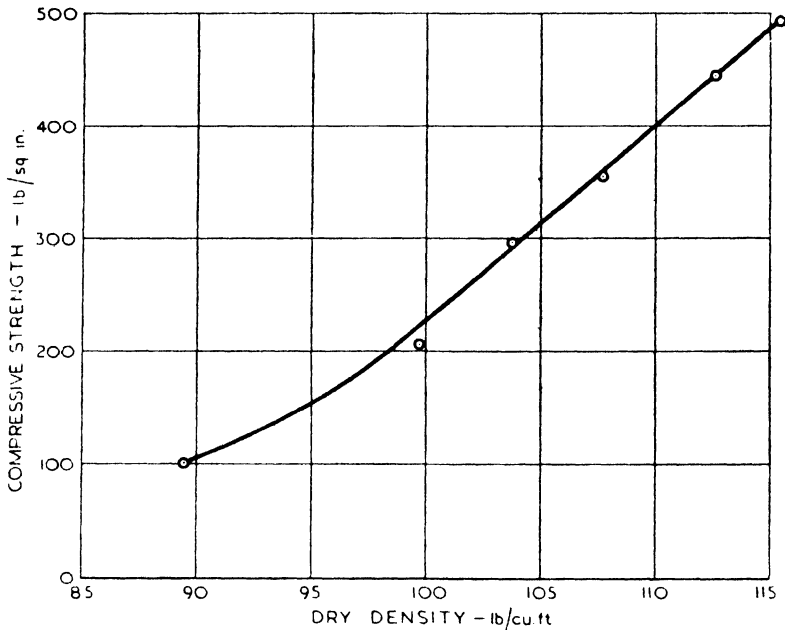



FIG. 12·3 TYPICAL RELATIONSHIP BETWEEN DRY DENSITY AND COMPRESSIVE STRENGTH OF 4-IN. SOIL-CEMENT CUBES

12·19 Laboratory tests made at the Road Research Laboratory on a silty clay soil have shown that for specimens having the same cement content and given the same amount of compaction, but having different moisture contents, the greatest strength is obtained at approximately the optimum moisture content. This confirms the view that the moisture content has a negligible effect on the quality of soil-cement, except in so far as it affects the compacted dry density.

Mixing

12·20 Although it is known that mixtures made in the laboratory have higher strengths and greater durability than similar mixtures made in the field, this factor is allowed for in the accepted standards for the preliminary tests. Work at the Road Research Laboratory indicates that the compressive strength of soil-cement made by the mix-in-place method with agricultural plant is about 40 to 60 per cent of that of corresponding laboratory mixes, while mixes made with an efficient rotary tiller have 60 to 80 per cent of the strength of a laboratory mix, and even higher percentages have been obtained with some soils.

Age and " Curing "

12·21 The compressive strength of a soil-cement mix increases with age in a similar manner to that of concrete. It is believed that the soil type affects the rate of hardening, but there is no definite information on this point. Fig. 12·4 illustrates typical strength/age relationships obtained in tests at the Road Research Laboratory for soil-cement made with a sandy clay soil.

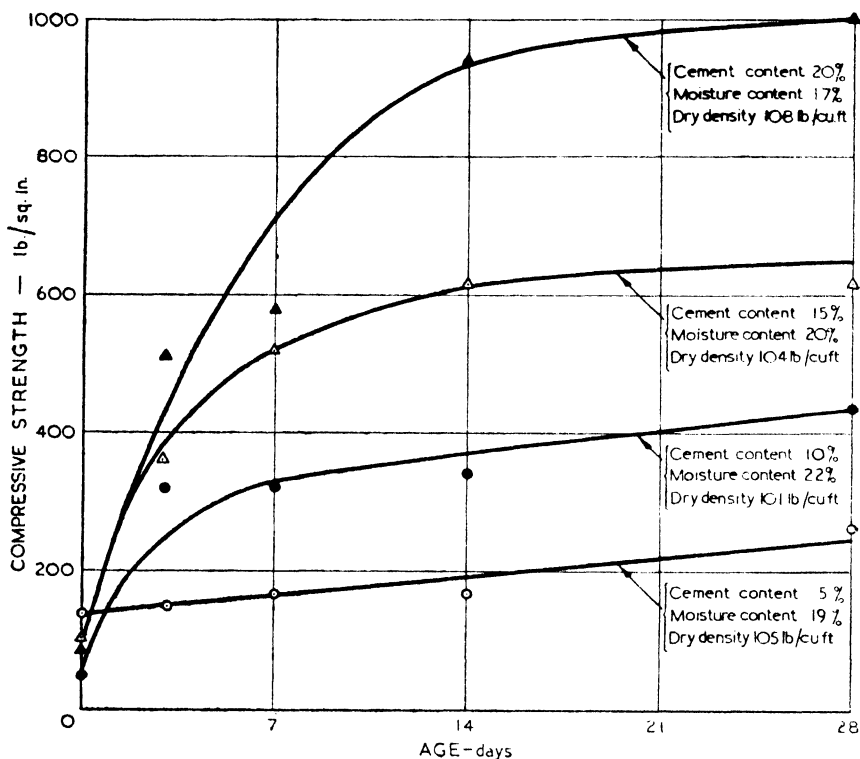



FIG. 12·4 STRENGTH/AGE RELATIONSHIPS FOR SOIL-CEMENT MADE WITH A SANDY CLAY WITH VARIOUS CEMENT CONTENTS AT OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY

12·22 In practice, soil-cement is " cured " after compaction under conditions that prevent drying of the surface during the initial period of the development of strength. The conditions under which this curing is carried out may affect the quality of the material produced, normally a damp atmosphere being desirable. However, such conditions do not always prevent the formation of a thin hard crust, which may crack and tend to flake off.

12·23 The application of a bituminous priming coat to the surface of soil-cement for curing purposes sometimes leads to slight surface disintegration, possibly because the penetration of the binder prevents hydration of the cement. The drying of soil-cement usually leads to the development of fine shrinkage cracks on the surface, but these are not believed to be iniurious.

Cement-modified Soil

12·24 A certain amount of study, both in the laboratory and in the field, has been carried out in the U.S.A. into the use of small quantities of cement to modify the properties of soil[5][6]. By this means a reduction in the liquid limit, plasticity index and water absorption may be obtained with clay soils. The idea has not found very much application in practice, but it illustrates an essential function of the soil-cement process. By bringing individual soil particles into aggregations, the effect of the cement is to increase the average particle size of the soil, thus conferring new properties on the soil that are independent of the structure of the soil-cement as a whole. In this connexion it has recently been reported that some soil-cement roads in Texas have been deliberately broken up three weeks after laying until the stabilized soil consisted of lumps of about 2-in. size, after which the roads were recompacted. It is understood that this procedure gives a better surface finish than the normal method of construction, since it enables shaping to be more easily carried out, and prevents cracking. These roads have been entirely satisfactory up to the present, although it is as yet too early finally to assess their durability.

CONSTRUCTION METHODS

12·25 The methods by which soil-cement mixtures may be produced in the field for road construction are described in detail in Chapter 15. Briefly, these methods are:—

Mix-in-place
Travelling plant
Stationary plant.

In this country and in the U.S.A. the mix-in-place method is the one most widely used, although travelling plant has been used frequently on large jobs, particularly in the U.S.A. During the recent war most of the plant used for mix-in-place in this country was agricultural equipment. Since the war, however, most soil-cement work has been carried out in place with modern rotary tillers. Most of the soil-cement construction of sub-bases for roads in this country has been done by stationary plant, however, and one county authority set up a central mixing plant for this type of work at one of its depots. The mix-in-place method appears to have been commonly used in many other countries, and soil-cement has been constructed in Germany with specially designed travelling plant. In England, stony cohesionless soils have also been mixed in concrete mixers of the standard type.

COSTS

12·26 The most recent figures available in 1950 for the cost of soil-cement construction in this country indicated a price of 3s. 6d. to 5s. 0d. per sq. yd including surface dressing, for work carried out by mix-in-place methods on areas less than about 30,000 sq. yd. Similar work on larger areas has cost about 2s. 0d. per sq. yd without a surface dressing. No figures are available for travelling plant stabilization, but the price is in general believed to be about the same as for the mix-in-place method.

12·27 Central plant mix methods are usually the most expensive, involving costs of about 5s. 0d. to 7s. 6d. per sq. yd including surface dressing, although in one case, under favourable conditions, the cost in 1947 was as low as 3s. 0d. per sq. yd without the surface dressing.

12·28 Detailed costs have been published by Warren[7] for the construction of 20,000 sq. yd of soil-cement by direct labour at Dartford in 1947. The mix-in-place method was used, and the total cost for the construction of a stabilized soil base 6 in. thick with a cement content of 8 per cent was 5s. 1½d. per sq. yd. Of this, 37 per cent was for hire of plant, 26 per cent for cement, 16 per cent for labour and transport involved in processing and 21 per cent for labour and materials used in applying a priming coat and a single surface dressing.

LABORATORY TESTING TECHNIQUES

12·29 The following laboratory tests are carried out to assess the suitability of a soil for stabilization with cement:—

Identification tests
Compaction test
Compressive strength test.

The following additional tests may be required in specific cases:—

Durability tests
Determination of the organic matter content
Determination of the sulphate content.

12·30 The compressive strength test is the most important so far as the design of the mixture is concerned, and the remaining tests can be regarded as ancillary. Tests such as those for water absorption, permeability or shrinkage are not normally required, since soil-cement of satisfactory quality as determined by the compression test will absorb little water and be relatively impermeable.

12·31 The purposes of the various tests, and the principal information that can be derived from them, are summarized below:—

Test		*Purpose and information obtained*
(1) Identification tests	(a)	To eliminate any poor soils, and to select those that are likely to be most suitable for further testing.
(2) Compaction test	(a)	To specify a suitable moisture content for field compaction.
	(b)	To specify a minimum dry density to be obtained in the field.
	(c)	To determine the moisture content to be employed in making compressive strength specimens.

Test		*Purpose and information obtained*
(3) Compressive strength test	(a)	To determine the suitability of the soil for treatment and to compare different mixtures.
	(b)	To specify the cement content to be used in the construction.
	(c)	To provide a standard by which the quality of the field processing can be assessed.
(4) Durability tests	(a)	To determine the suitability of certain border-line soils for stabilization—this test is only used occasionally.
	(b)	To investigate the suitability of stabilized soil for use under particularly severe climatic conditions.
(5) Determination of sulphate and organic matter content of the soil	(a)	To assist in the preliminary selection of soils suitable for stabilization in areas where high organic matter or sulphate contents are suspected.
	(b)	To establish the reason for the unsuitability of a soil.

Identification tests

12·32 Particle-size analyses and plasticity tests are made on the natural soil according to the procedures in B.S. 1377·1948, which are described in detail in Chapter 3.

12·33 By means of these tests, the soils suitable for further testing can be selected from the samples obtained in a soil survey. Soils which fall well outside the limits quoted in the section on " soil type " in this chapter can be rejected immediately as being unsuitable for stabilization. In making a choice between a number of soils, all of which may be suitable for processing, preference should be given to those with a good particle-size distribution (see Chapter 11).

12·34 The results of the plasticity tests can be used to indicate the ease with which soils may be mixed in the field. Slightly conservative estimates of the probable cement content required are sometimes advisable when the plasticity index of a soil tends to be high, and when the mixing machines available are of an inferior type. The possibility of mixing certain non-plastic coarse-grained soils in ordinary concrete mixers can also be judged from the results of the identification tests.

12·35 These tests also give an approximate indication of the surface texture of the finished soil-cement, and from this the probable resistance to surface abrasion by traffic can be inferred.

Determination of the Organic Matter Content of the Soil

12·36 The organic matter content of soil is determined by the dichromate oxidation method described in Chapter 5. An indication of a high organic matter content may also be obtained during the treatment of the soil with hydrogen peroxide prior to the particle-size analysis, as described in Chapter 3.

Determination of the Sulphate Content of the Soil

12·37 This is only carried out when a relatively high salt content in the soil is suspected. The procedure employed is that described in Chapter 5.

Compaction Test

12·38 When dealing with soils containing not less than 80 per cent of particles passing the ¾-in. B.S. sieve (fine- and medium-grained soils), this is carried out with the apparatus used in the B.S. compaction test (see Chapter 9), the main difference in procedure being that a fresh sample is used for each determination of dry density at a different moisture content. With soils containing not less than 80 per cent of particles passing the 1½-in. B.S. sieve (coarse-grained soils), the test may be carried out with a mould having an internal diameter of 6 in. and 12 in. high, fitted with a detachable collar. The soil-cement is compacted into this mould in 6 approximately equal layers, using 25 blows of a 10-lb. rammer falling through a height of 18 in. on to each layer. With both types of soil the oversize material is rejected for the test.

12·39 A large sample of the soil to be tested (15-20 kgm) is air-dried, and the air-dried moisture content determined. The soil is then quartered or passed through a riffle until eight batches, each of about 2 kgm, are obtained. Suitable quantities of water are then added to the soil to enable a range of moisture contents covering the probable optimum moisture content to be obtained. The moisture contents aimed at should be preferably 1·5 to 3 per cent apart. The water is then mixed in, and the damp soil stored for 24 hours in an airtight container, to enable the moisture to become uniformly distributed throughout the soil.

12·40 The required proportion of cement is then added and mixed in with an efficient type of laboratory mixer. For sandy soils 8 per cent of cement is suitable, for well graded sandy clays 9 to 10 per cent and for clays 10 to 12 per cent of cement is usually required, based on the dry soil weight. Each batch of soil is then compacted into the standard mould in the manner employed in the test with unstabilized soil. The dry density is determined in the usual way, and the moisture contents found by oven-drying samples at 105 to 110°C. and expressed in terms of the oven-dry weight of soil-cement. The final dry density obtained is expressed in pounds per cubic foot of dry soil-cement.

12·41 The dry density/moisture content relation of the soil-cement may also be obtained from field tests, made at different moisture contents with the compaction plant that is to be used in the construction. The dry density obtained during the actual construction should not be less than 95 per cent of the maximum dry density indicated by either the field or the laboratory tests. The possible differences that may arise between the results of a compaction test carried out in the laboratory and of compaction trials carried out in the field are discussed in Chapter 9.

12·42 It is particularly helpful to carry out compaction trials in the field when dealing with mixtures made with uniformly graded sands that are subsequently to be compacted by vibratory methods, since the optimum moisture contents obtained by such methods are significantly lower than those

indicated by the B.S. compaction test. Preliminary compaction trials in the field are desirable with all soil-cement mixtures, and should be carried out whenever practicable.

12·43 As an example of the difference between laboratory and field tests, a sand tested at the Road Research Laboratory had an optimum moisture content of 11 to 12 per cent and a maximum dry density of 112 lb./cu.ft in the B.S. compaction test, when a 5-per cent admixture of cement was used. At this moisture content, however, the soil-cement was nearly saturated, and experience with concrete work suggested that better compaction would be obtained at lower moisture contents using vibration methods. Field compaction trials were accordingly carried out with a vibrator, and the results showed an optimum moisture content of 6 per cent and a maximum dry density of 114 lb./cu.ft. Fig. 12·5 shows the two curves obtained, together with the results of other compaction tests illustrating how the maximum dry density varied with change of cement content.

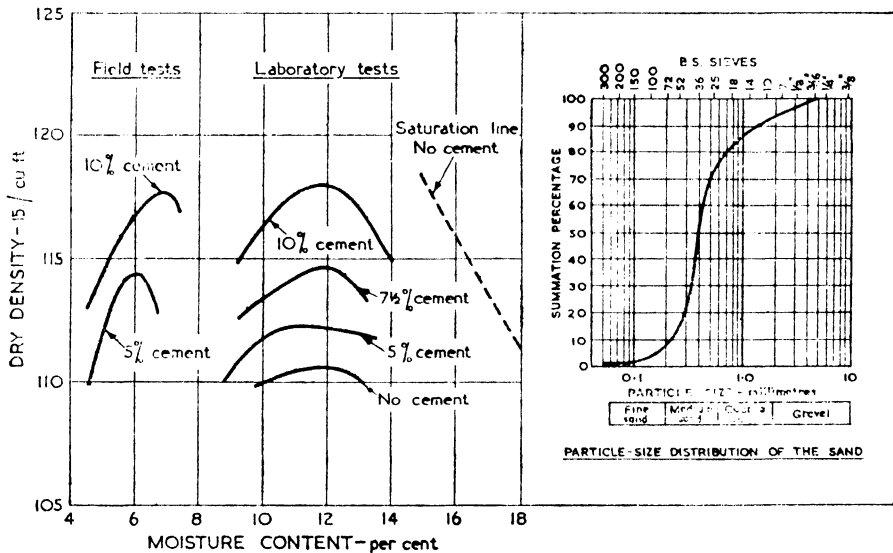



FIG. 12·5 COMPARISON OF RESULTS OF FIELD AND B.S. COMPACTION TESTS ON CLEAN SAND MIXED WITH VARIOUS PROPORTIONS OF CEMENT

12·44 In the case quoted above, the soil had a deficiency of particles in the silt range, and the cement, by making up this deficiency, increased the resulting dry density (see Chapter 11). Fig. 12·6 illustrates the results obtained in laboratory tests on a soil containing an appreciable proportion of particles in the fine sand and silt range. In this case the addition of the cement caused a decrease of dry density. As the cement content was increased, however, its higher specific gravity (3·15 as against 2·65 for soil) overcame the tendency of the poorer size-distribution to produce a decrease of dry density.

FIG. 12·6 RESULTS OF B.S. COMPACTION TESTS ON A SANDY CLAY MIXED WITH VARIOUS PROPORTIONS OF CEMENT

The Compressive Strength Test

12·45 This is usually carried out with cylindrical test specimens having a height:diameter ratio of 2. Three sizes of specimen are employed, depending on the particle-size distribution of the soil, i.e.:—

Fine-grained soils containing not less than 80 per cent of particles passing a No. 7 B.S. sieve:—Diameter = 2 in. Height = 4 in.

Medium-grained soils containing not less than 80 per cent of particles passing a $\frac{3}{4}$-in. B.S. sieve:—Diameter = 4 in. Height = 8 in.

Coarse-grained soils containing not less than 80 per cent of particles passing a $1\frac{1}{2}$-in. B.S. sieve:—Diameter = 6 in. Height = 12 in.

The sizes used should be the smallest of the three, consistent with the above limitations.

12·46 Specimens are prepared in a constant volume mould, in which circular pistons may be pressed in until they are a fixed distance apart. By using a calculated weight of material, the final dry density of the specimen is adjusted

to the maximum value obtained in the compaction test. Specimens 4 in. cube may also be prepared, either in a constant-volume mould (Plate 12·1), or in an ordinary concrete cube mould if the latter is available in connexion with other work.

12·47 A suitable quantity of soil is air-dried and its air-dry moisture content determined. The amount of material required for one cylindrical specimen of each of the three sizes given in paragraph 12·45 are 400-600 gm, 1,500-2,500 gm and 7,000-9,000 gm respectively, assuming that oversize material, i.e. stones retained on the No. 7, ¾-in. or 1½-in. B.S. sieves, has already been removed. The soil is then mixed thoroughly in a laboratory mixer with sufficient water to bring the moisture content to a value 3 per cent below the optimum moisture content for the soil-cement mixture as determined in the compaction test. The damp soil is then stored in an airtight container for 24 hours to enable the moisture to become uniformly distributed throughout the soil. The required amount of cement, calculated as a percentage of the dry soil weight, is then mixed in by the laboratory mixer for about 1 to 2 minutes, and the remaining 3 per cent of water is added and mixing continued for a further 8-9 minutes.

12·48 Where specimens are made in constant-volume moulds, the weight of soil-cement mixture required to fill the mould is calculated from the known final volume of the specimen and the value of the maximum dry soil-cement density obtained in a compaction test with a similar mixture. This weight of material is then placed in the mould, and the pistons are pressed home and held in position for a few seconds. With most cohesive soils, the specimens can be de-moulded immediately, care being taken to avoid damaging the corners. With sandy soils, it may be necessary to drive home the pistons with a vibrating hammer, and such specimens should be left in the mould for 24 hours to enable some hardening to take place prior to de-moulding.

12·49 In preliminary testing 10 2-in. diameter specimens are made at each cement content; five are normally prepared when 4-in. or 6-in. diameter specimens are used. All three types of specimen are cured by coating the surfaces with a layer of paraffin wax, to keep the moisture content constant, and storing them for seven days in a room where the temperature is fairly uniform. After curing the wax coating is removed from the flat faces, and the specimens are tested in compression at a rate of 250 lb./sq.in./min., or at a rate of strain of about 0·05 in./min. The moisture content of the specimen is determined after testing, using the fragments, and the dry soil-cement density checked by weighing and calculation. The average values of compressive strength, dry soil-cement density and moisture content are reported for each batch of five or ten specimens. A compressive strength of 250 lb./sq. in. after seven days is usually accepted as a satisfactory minimum in this country, using normal Portland cement. Under more severe climatic conditions a higher compressive strength may be desirable, e.g. 400-500 lb./sq.in.

12·50 If moulds for making concrete test cubes are the only testing apparatus available, the following procedure can be adopted for preparing specimens. Sufficient soil-cement, prepared as described in paragraph 12·47, is placed in the mould to fill it approximately one-third full when compacted. A cube of wood of slightly less than 4-in. side is then placed on the mixture, and given 28 blows with the 5½-lb. rammer used in the B.S. compaction test, dropped

through a height of 1 ft (see Chapter 9). The mould is filled with two additional layers of soil-cement of the same thickness in a similar manner, using a wooden collar for the third layer so that finally the compacted material is slightly proud of the top of the mould. As each layer is compacted, the smooth surface is scarified with a pen-knife or spatula, to enable the next layer to make a proper bond with the material below. The soil-cement is struck off level with the top of the mould with a straight-edge, and the specimen is de-moulded when it has enough cohesion.

Durability Tests

12·51 These were originated by the Portland Cement Association of America and are fully described in one of their handbooks[8]. Four soil-cement cylinders are made up at each of several cement contents, at the optimum moisture content, in the manner used in the compaction test, and two are submitted to cycles of wetting and drying and two to freezing and thawing.

12·52 For the wetting and drying test, after 7 days' curing in a damp atmosphere, the specimens are immersed in water for 5 hours, after which the No. 1 specimen is weighed and measured, and both specimens are placed in an oven at 160°F. for 42 hours. On removal, both specimens are weighed, the No. 1 specimen is measured, and the No. 2 specimen brushed firmly all over with a stiff wire brush (a specified method is given, using a standard brush), and weighed again. This is repeated until the specimens have been through twelve cycles of wetting and drying.

12·53 For the freezing and thawing test, after 7 days' curing the specimens are placed on water-saturated felt pads or blotters and stood on carriers in a refrigator at a temperature not higher than −10°F. for 22 hours. On removal the No. 3 specimen is weighed and measured, and both specimens are then kept in a moist atmosphere at room temperature for 22 hours, care being taken that the felt pads are kept moist. Both specimens are then weighed, the No. 3 specimen measured, and the No. 4 specimen brushed and weighed as for the No. 2 specimen. This is repeated until the specimens have been through 12 cycles of freezing and thawing. The maximum swelling of the Nos. 1 and 3 specimens is calculated on the basis of the volume as moulded, and the loss in weight of the Nos. 2 and 4 specimens is found on the basis of the oven-dry weight. The maximum allowable swelling is 2 per cent, and the maximum loss of weight is as shown in Table 12·4 below, which also summarizes the effect of cement on various soils.

12·54 It is felt that these durability tests are rather severe, and correspond more to American weathering conditions than to those in the British Isles. Consequently, a soil-cement mixture that will pass these tests should be suitable for use in this country.

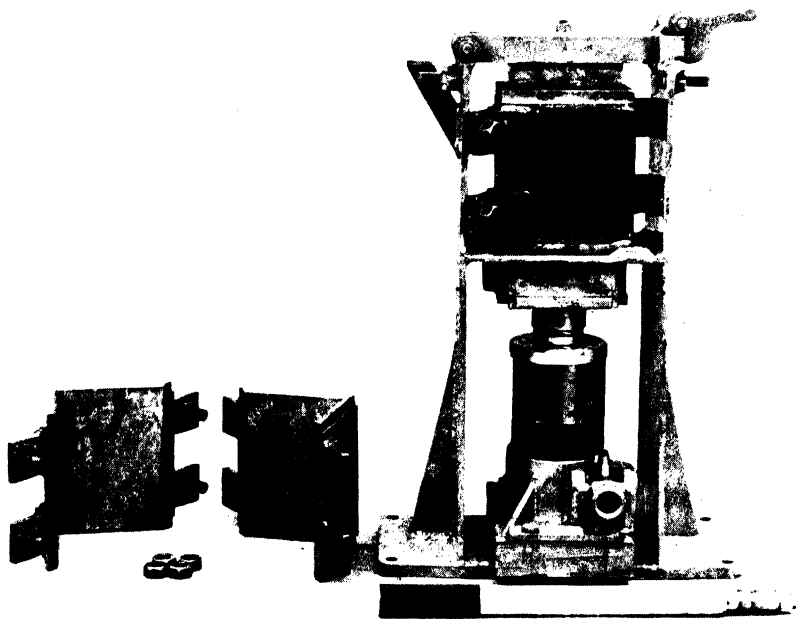
12·55 As a general guide, it may be mentioned that a large number of such tests made at the Road Research Laboratory show that a soil-cement mixture having a good compressive strength will usually also show a good resistance to damage in the durability tests. Exceptions to this rule have been encountered, however, particularly in mixtures with some very clayey soils and

CONSTANT-VOLUME MOULD FOR MAKING 4-IN.
SOIL-CEMENT CUBES

PLATE 12·1

certain shales, and it is also believed that the wetting/drying test may be of some additional value when dealing with soils from tropical and sub-tropical areas.

TABLE 12·4

SOIL-CEMENT GROUPS

Soil type					
	P.R. Classification	A2, A3	A4, A5	Some A6, A7	A8, some A6, A7
	Casagrande Classification	GF, SF, GW, SW, GP, SP	ML, CI, CL, MH	ML, CI, CL, CH	Pt, OH, OL, CH
		Gravelly or sandy soils	Silty soils	Clay soils	Peaty and highly organic soils, heavy clays
Effect of cement		Show very marked hardening	Show marked hardening	Show substantial hardening	Cannot be successfully treated
Minimum acceptable compressive strength		250 lb./sq. in.	250 lb./sq. in.	250 lb./sa. in.	—
Maximum allowable volume change in durability tests		2%	2%	2%	—
Maximum allowable loss of weight in durability tests		14%	10%	7%	—
Normal cement content		6—10%	8—12%	10—14%	—

FIELD CONTROL TESTS

12·56 The following tests are of assistance in controlling the quality of soil-cement produced in the field:—

Determination of the moisture content of soil and of soil-cement.

Compressive strength test.

Determination of the dry soil-cement density *in situ.*

Determination of the cement content.

Determination of Moisture Content

12·57 This may conveniently be done either on the soil or on the soil-cement mixture by the sand-bath method or by the pycnometer method, both of which are described in detail in Chapter 3.

Compressive Strength Test

12·58 This may be done by taking samples of the mixed soil-cement immediately prior to compaction and moulding them into cylindrical test specimens of a size appropriate to the particle-size distribution of the soil (see paragraph 12·45).

12·59 With soil containing not less than 80 per cent of particles passing a No. 7 B.S. sieve, sufficient of the fresh soil-cement mix is placed in a 2-in. diameter mould to give a specimen 4-4½ in. long after compaction. Compaction is then applied to the soil through a sliding metal plunger resting on the soil, which is subjected to 12 blows of the rammer used in the B.S. compaction test, dropped through a height of 12 in. The mould and specimen are then reversed and the opposite face of the specimen is compacted similarly with a further 12 blows from the rammer. The compacted specimen is then extended until only 4 in. of its length remain in the barrel of the mould, and the surplus length is trimmed off. After extension, the specimen is coated with wax, cured and tested in the manner described in paragraph 12·49.

12·60 With soil containing not less than 80 per cent of particles passing a ¾-in. B.S. sieve, a mould having an internal diameter of 4 in. and a height of 8 in., and fitted with a detachable collar is used. The fresh soil-cement is compacted into this mould in 6 equal layers, 25 blows of the rammer employed in the B.S. compaction test being applied to each layer. The amount of soil-cement used in these layers should be sufficient to give a specimen with a compacted height of just over 8 in., surplus material being trimmed off with a spatula. The ejected specimen is coated with wax, cured, and tested in the manner previously described.

12·61 The procedure with soils containing not less than 80 per cent of particles passing a 1½-in. B.S. sieve is similar, but with the difference that the mould required has an internal diameter of 6 in. and a height of 12 in. The soil is compacted in 6 equal layers, using 25 blows of a 10-lb. rammer falling through a height of 18 in.

12·62 With all three types of soil, the fresh soil-cement is first passed through the appropriate B.S. sieve (i.e. No. 7, ¾-in. or 1½-in.) and the oversize material rejected.

Determination of the Dry Soil-Cement Density in Situ

12·63 This can be done by the sand-replacement method, which is fully described in Chapter 9. The dry density should be measured over the full depth of processing, and when the latter is greater than 6 in. the measurement can be done in two stages and the average value taken, e.g. from the surface to a 5-in. depth and from a 5-in. to a 10-in. depth in the case of a 10-in. depth of treatment. As indicated in Chapter 9, the depth of the calibrating can used should be the same as the depth of the excavated hole in an individual measurement. The determination of the dry density also provides an accurate check on the depth of processing, which should be recorded simultaneously.

Determination of Cement Content

12·64 This is done by a chemical method, in which samples of the soil, cement and soil-cement mixture are treated with hydrochloric acid, and the calcium contents of the resulting solutions are determined. Since the method requires a fairly well equipped laboratory and a skilled analyst, it is not suitable for general field control work, and is usually applied only in an analysis of the finished work.

SUMMARY

12·65 Soil stabilization with cement has been used on an increasing scale for the construction of roads and airfields since its introduction in the U.S.A. in 1935. Most coarse-grained soils and many fine-grained soils with clay contents not exceeding about 30 per cent can be processed successfully, but soils containing organic matter or deleterious chemicals are generally unsuitable. Normally, an increase in cement content produces a material of better quality. The moisture content of the processed soil affects both the uniformity of mixing and the compaction obtained. In practice a moisture content somewhat higher than the optimum for maximum compaction is most suitable. Good compaction is essential to obtain a material of high quality, and great care must be taken in this respect in the field. The uniformity with which the cement is mixed into the soil affects the quality of the soil-cement; the most efficient methods of field mixing yet studied do not give a material having a strength exceeding 80 per cent of the strength that can be obtained with the same material by laboratory mixing.

12·66 In this country a compressive strength test is used to determine the suitability of a soil for processing with cement. In the U.S.A., durability tests are used in which samples of the soil-cement mixture are either successively wetted and dried or frozen and thawed.

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CHAPTER 13

STABILIZATION OF SOIL WITH BITUMINOUS MATERIALS

INTRODUCTION

13·1 This chapter is concerned with the group of processes in which soils are stabilized by the addition of bituminous materials, such as asphaltic bitumen, tars or emulsions. Although more general reference is made to bitumens, tars are equally suitable and, except when the use of bitumens of very low viscosity is prescribed, it may be taken that a tar of equivalent viscosity may be used under similar conditions. As with the other uses of tar more careful control is required than in the case of bitumen.

13·2 In temperate climates the moisture content of cohesive soils is fairly high during most of the year and the addition of further fluids in the form of bituminous materials may cause a loss in strength. For this reason bituminous soil stabilization has not been widely employed in the British Isles. In hot dry climates the soil is often at a low moisture content and there is a correspondingly larger field for the use of fluid stabilizing agents.

13·3 The main use of bituminous-stabilized soil for roads has been in the construction of bases for lightly trafficked surface-dressed roads. For some main roads having concrete or bituminous surfacings, bituminous materials have been used to waterproof the subgrades and thus preserve their stability.

TYPES OF BITUMINOUS STABILIZATION

13·4 When a bituminous material is mixed with soil it may either bind the particles together or it may waterproof the soil, thus preserving the bonding action of water films between the particles, or both these effects may occur together. It is convenient to discuss the various methods employed in bituminous soil stabilization under these two headings.

Bituminous Materials as Binding Agents

13·5 OILED EARTH ROADS. For certain areas of the world such as the Middle East, and the middle western states of the U.S.A., the natural moisture content of some of the silty and clayey soils in the dry season is low enough to permit the unsurfaced soil to carry traffic. Such earth roads, which suffer from the disadvantages that they dust very easily and soften during the rainy season, can be improved at a relatively low cost by spraying the dry soil surface with an oil or with a medium-curing cut-back bitumen having a viscosity in the range 3 to 30 sec. S.T.V. (MC.1 or MC.2) at 25°C. The stabilizer is usually applied in two or three approximately equal distributions totalling about 1 gal./sq.yd so that it penetrates ½ to 1 in. into the soil.

13·6 The amount of penetration depends largely on the moisture content of the soil at the time of application. If the soil is dry a film of dust may be formed on the surface through which the oil or bitumen cannot penetrate, while if the soil is too wet the pores will be filled with water leaving no spaces for the oil to penetrate. A preliminary wetting or drying may therefore be required.

13·7 Roads of this type become rather slippery during rainy weather and it may therefore be desirable to blind the surface with a light dressing of sand if it is available.

13·8 THE USE OF CUT-BACK BITUMENS. The stabilized soil crust of an oiled earth road has to withstand the combined effects of weather and traffic. When these effects are severe the surface skin is insufficient and it becomes necessary to treat a greater depth of the soil, for instance by mixing in a bituminous material by means of suitable plant.

13·9 The incorporation of a material such as a cut-back bitumen into soil increases the total fluids content: if this is already fairly large by virtue of a high moisture content, lubrication of the particles will take place and the soil will become unstable and plastic. For this reason it is usually not practicable to add relatively large quantities (7 to 10 per cent) of bitumen to cohesive soils in this country since the climate is such that they have an appreciable moisture content during most periods of the year.

13·10 The use of these materials may however be advantageous in certain tropical and arid climates where the soils may be very dry. In these circumstances the bituminous binder will provide a large portion of the optimum fluids content that is required for maximum compaction: the resulting economy in water may be an important factor.

13·11 The immediate effect of the addition of a cut-back bitumen on the properties of a sandy clay soil is indicated in Figs. 13·1, 13·2 and 13·3. Fig. 13·1 shows that increasing amounts of bitumen cause the maximum dry density obtained in the B.S. compaction test (see Chapter 9) to be progressively reduced, presumably owing to the greater viscosity of the fluid films surrounding the particles, while a corresponding increase in the optimum fluids content is observed. Fig. 13·2 shows the effect of different proportions of the same bitumen on the unconfined compressive strength of specimens of the same soil-bitumen mixtures made up at the maximum dry densities obtained with the B.S. compaction test. The strength of the specimens rises slightly until a binder content of 4 per cent is reached, after which it decreases progressively until it finally reaches a value lower than that of the untreated soil. This suggests that the binding effect of the bitumen serves initially to increase the strength but beyond a certain point the decrease in dry density due to the resistance of the mixture to compaction offsets the increase in cohesion.

13·12 Fig. 13·3 shows the amount of water absorbed in the capillary water absorption test (see " Waterproofing Tests " para. 13·53) by specimens of the sandy clay soil with the same range of stabilizer contents and similar dry densities. The addition of 2 per cent of bitumen causes an increase in the water absorbed after 28 days, presumably owing to a reduction in the dry

density, but quantities of 4 per cent and over cause a substantial reduction in the water absorbed.

13·13 Taking into account the results of the compaction, compressive strength and water absorption tests illustrated in Figs. 13·1, 13·2 and 13·3, the best results in the field would probably be achieved with this soil by the addition of 4 per cent of the particular bitumen tested. This would give a mixture having optimum compaction and strength characteristics and adequate resistance to water absorption. The use of more bitumen would be unnecessary and in extreme cases might be harmful since the compacted dry density and hence the strength would be considerably reduced.

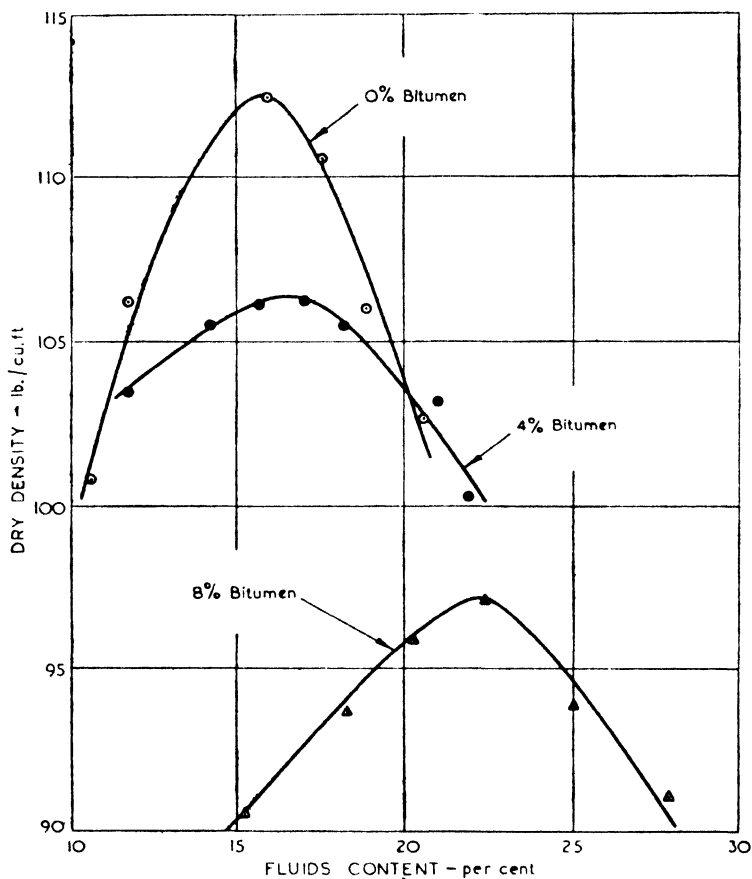



FIG. 13·1 RESULTS OF B.S. COMPACTION TESTS ON A SANDY CLAY CONTAINING VARIOUS PROPORTIONS OF CUT-BACK BITUMEN

13·14 The performance of bituminous-stabilized soil may also be affected by the distribution of the cut-back bitumen between the soil particles. Thus Benson and Becker[1] have shown that the stability of a soil/water/cut-back bitumen mixture prepared in a given mixing machine is a function of the time of mixing. The stability rises to a maximum after a certain period of mixing,

after which further mixing causes a decrease. In this respect bituminous materials differ from other stabilizing agents in that it may not be desirable, either in the laboratory or in the field, to seek to obtain the most intimate possible mixture.

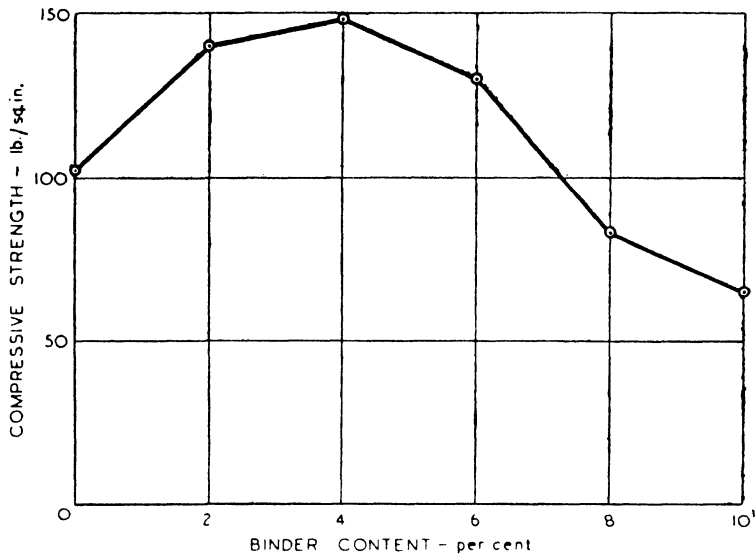



FIG. 13·2 RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND BINDER CONTENT FOR SPECIMENS OF A SANDY CLAY CONTAINING CUT-BACK BITUMEN

13·15 In suitable climatic conditions a variety of soils may be stabilized with cut-back bitumen, but as with most other forms of stabilization the upper limit of clay content in the soils that can be dealt with is to some extent determined by the inability of existing plant to pulverise very clayey soils and mix stabilizers into them. The best results are obtained when reasonably well graded soils are used, and the Highway Research Board of America[2] suggests the limits given in the following list:—

Maximum size of material—not greater than approximately one-third the compacted thickness.

Passing:—$\frac{3}{16}$-in. B.S. sieve — > 50 per cent
No. 36 B.S. sieve — 35-100 per cent
No. 200 B.S. sieve — 10-50 per cent

Liquid limit < 40 per cent. Plasticity index < 18 per cent.

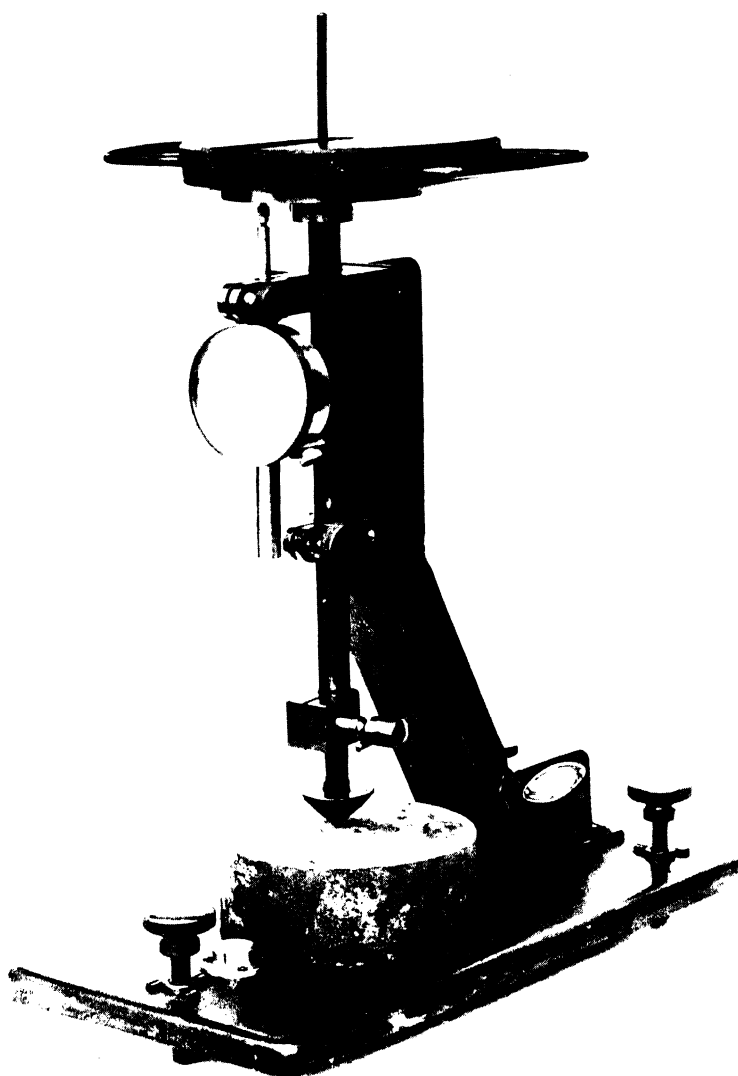
13·16 It is very difficult to stabilize cohesive soils with bitumen in this country, owing to the relatively high moisture contents. With sands, however, there is no such difficulty since excess water may be squeezed out by rolling provided that excessive quantities of bitumen are not added. The " wet-sand-mix " process was widely employed in this country during the recent war for airfield construction. In this process 4 to 10 per cent of a cut-back bitumen

CONE PENETROMETER

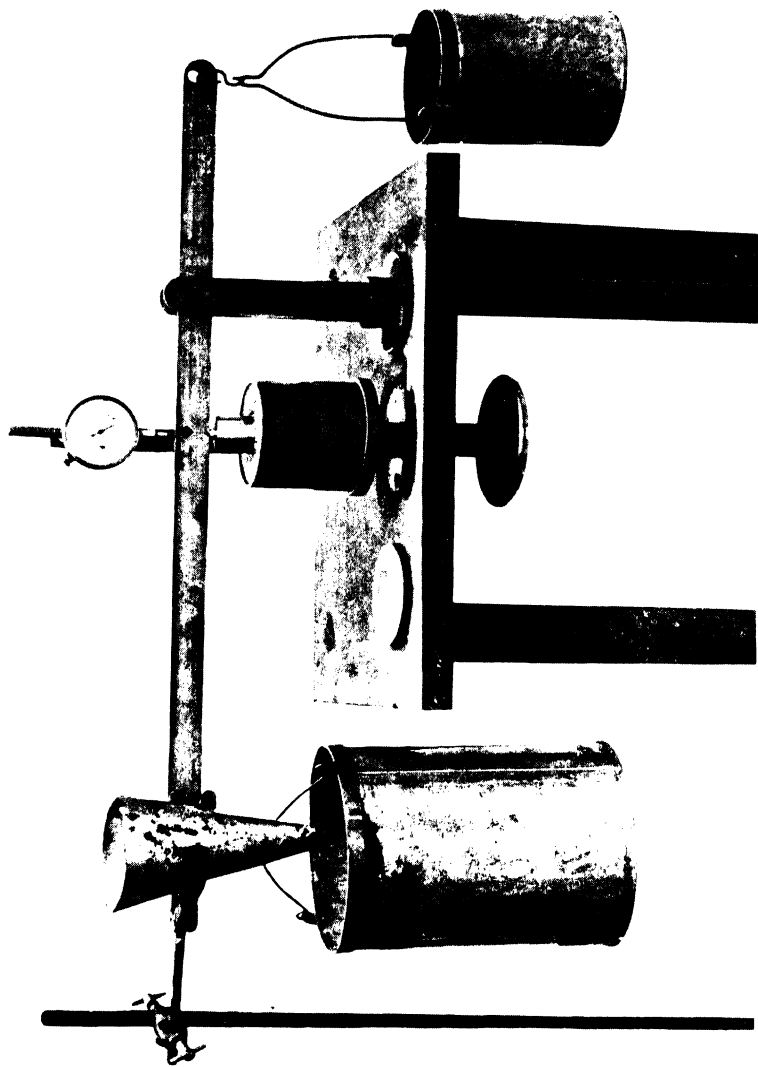
PLATE 13·1

FLORIDA BEARING VALUE APPARATUS

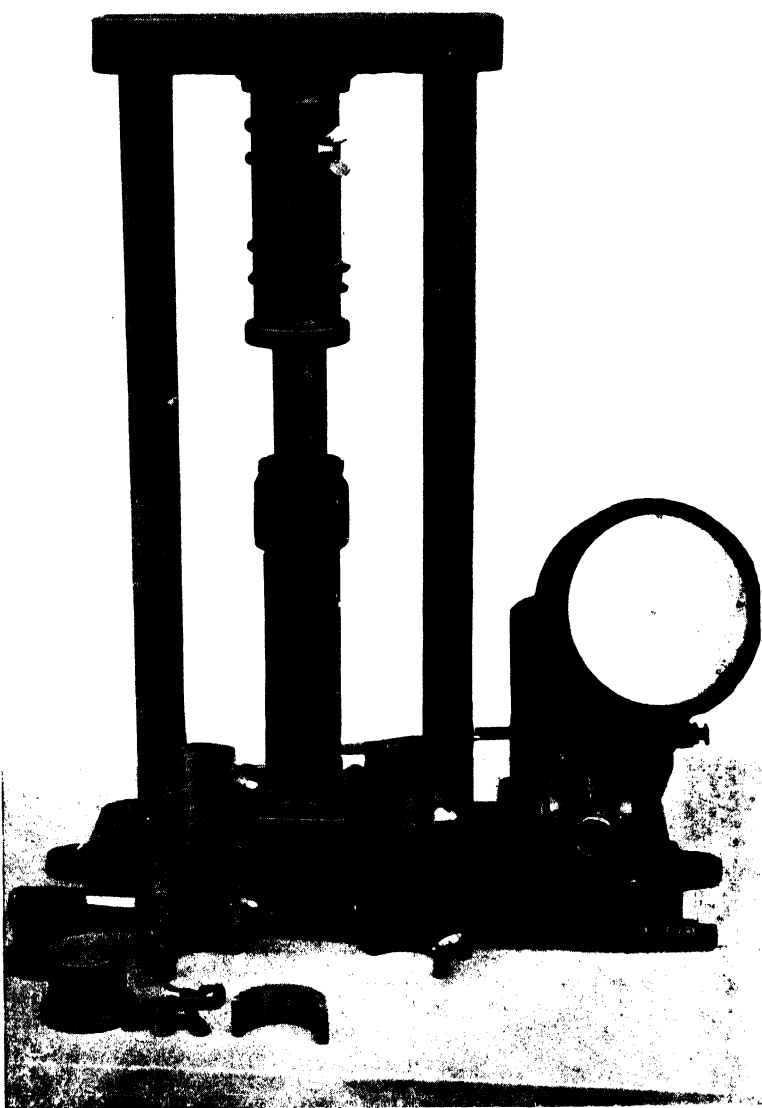
PLATE 13·2

APPARATUS FOR MAKING SPECIMENS FOR THE CAPILLARY WATER ABSORPTION TEST

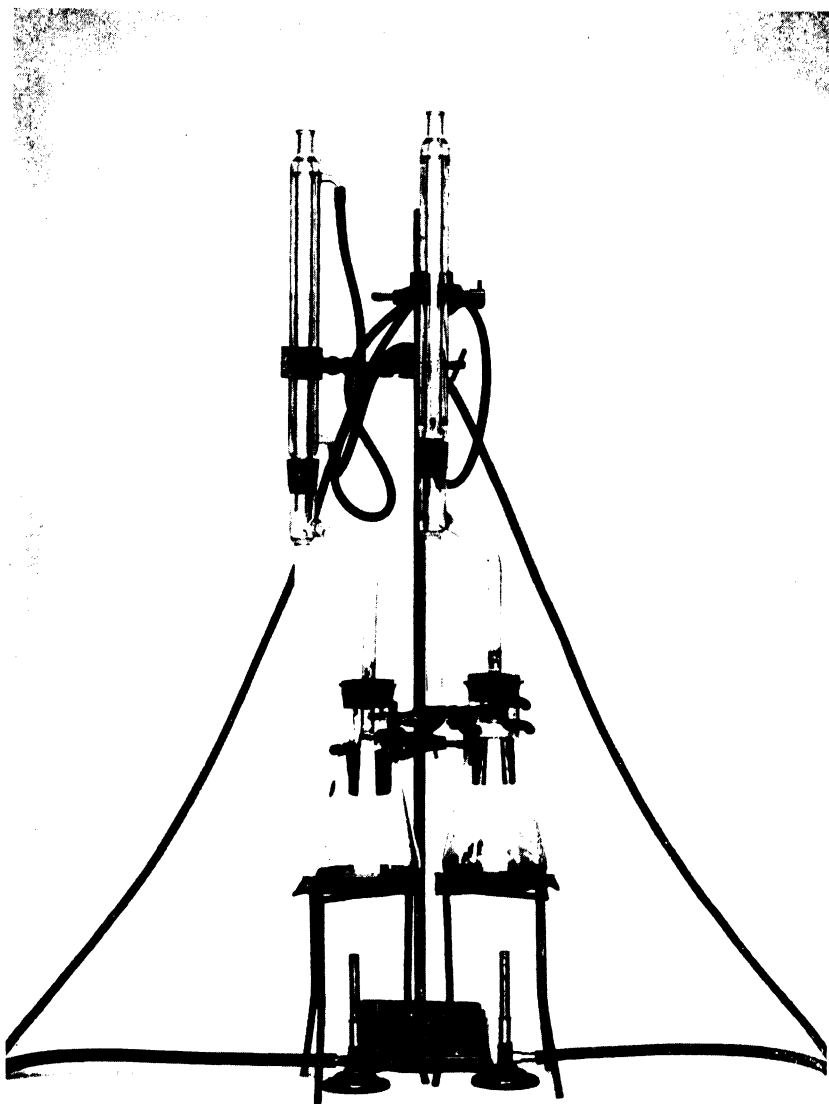
PLATE 13·3

SOLVENT DISTILLATION APPARATUS
for the determination of the moisture content of bituminous-stabilized soil

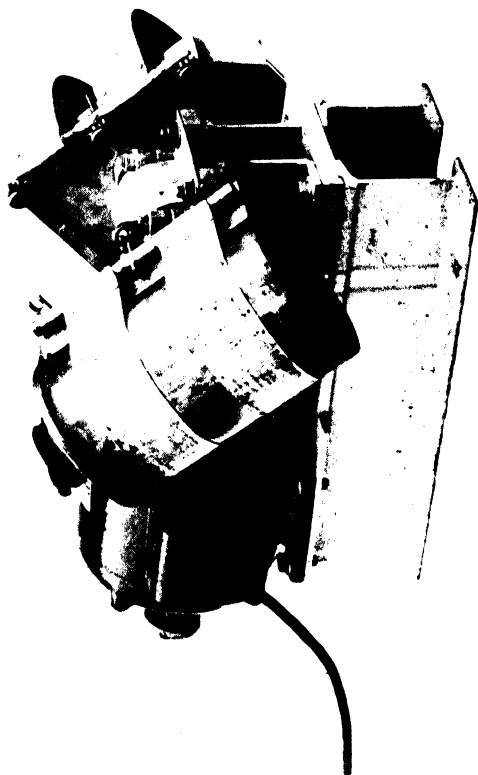
PLATE 13·4

MECHANICAL SHAKER
for use in cold extraction method of estimating the stabilizer content of bituminous-stabilized soil

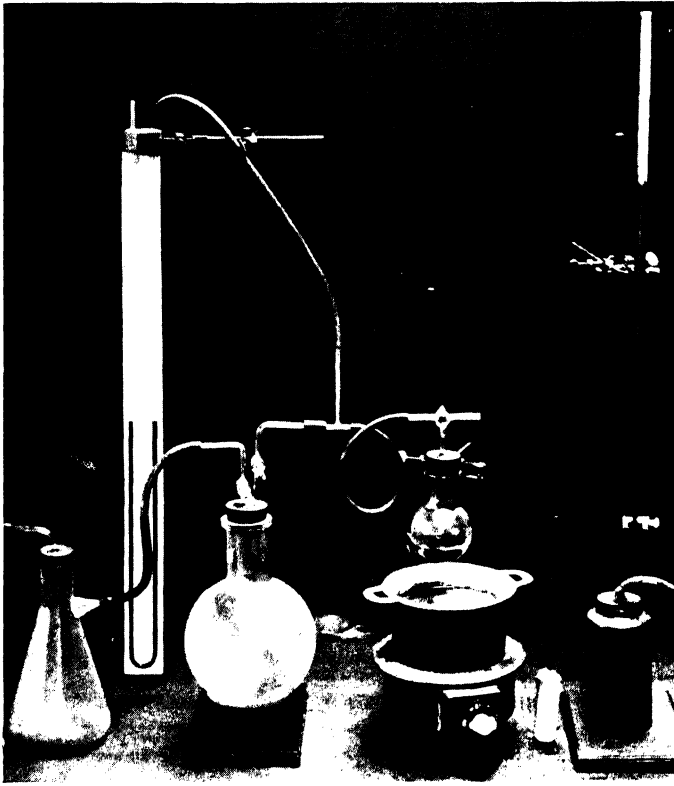
PLATE 13·5

APPARATUS REQUIRED IN COLD EXTRACTION METHOD
for estimating the stabilizer content of bituminous-stabilized soil

PLATE 13·6

is mixed with the wet sand to which 1 to 2 per cent of hydrated lime has been added to assist coating. The method is an extension of the " dry-sand-mix " process which has been widely used for road and airfield construction in the countries of the Middle East.

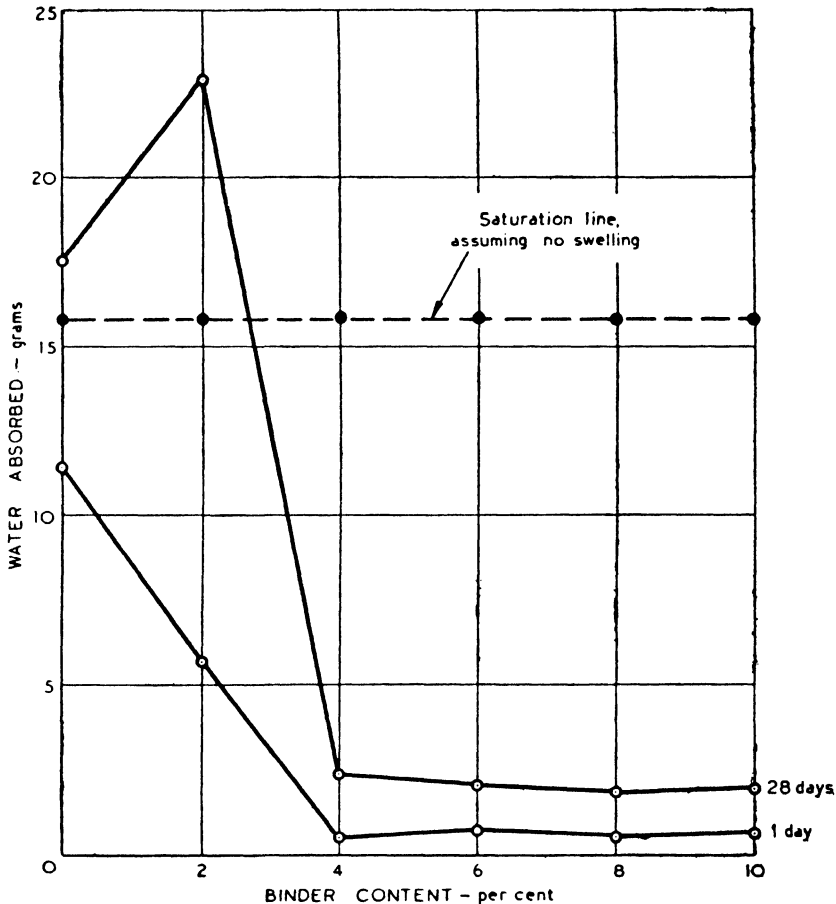



FIG. 13·3 RELATIONSHIP BETWEEN WATER ABSORPTION AND BINDER CONTENT FOR SAMPLES OF SANDY CLAY CONTAINING CUT-BACK BITUMEN

13·17 A wide range of sands may be used in the wet-sand-mix process and the particle-size distribution does not appear to be critical although a well graded sand will have a higher stability than one that is single-sized. Experience suggests that in this country not more than 5 per cent of material passing the No. 200 B.S. sieve can be tolerated, otherwise the mixture will become spongy and difficult to compact. On the other hand, in the U.S.A. where sand-mix roads are popular in the warm southern states, the recommended practice allows the use of sand containing 12 per cent of material passing the No. 200 B.S. sieve and when wind-blown or dune sand is used the figure may be increased to 25 per cent.

13·18 The types of cut-back bitumen most commonly used for soil stabilization are the medium-curing types with viscosities in the range 3 to 30 sec. S.T.V. at 25°C., i.e. MC.1, MC.2 grades. (See the appendix to this chapter for British equivalents of American grades of bitumen and tar.) However, both rapid-curing and slow-curing cut-back bitumens of similar viscosity (RC.1, RC.2, SC.1 and SC.2) have also frequently been used.

13·19 For sand mixes the characteristics of the cut-back bitumens used will depend on the particle-size distribution of the sand, the temperature of application and the type of mixing plant. Viscosities in the range 3-140 sec. S.T.V. at 25°C. have been successfully employed in the U.S.A., and the rapid-curing grades are most commonly used. The more viscous binders are used with sands having only a small proportion of material passing the No. 200 B.S. sieve and for plant mixes, while the lighter binders are used for mix-in-place methods and with soils containing a larger proportion of fines.

13·20 THE USE OF EMULSIONS. Bituminous emulsions are generally only suitable for soil stabilization in climates where rapid drying conditions occur, since this is equivalent to adding water to the soil as well as a bituminous binder. In the British Isles this tends to make cohesive soils plastic since in the natural state they often have a relatively high moisture content. In dry conditions however, such as in the Middle East and parts of Africa and India, the water in the emulsion may be an advantage since it helps to provide part of the optimum fluids content for compaction, thereby reducing the amount of water necessary for this purpose.

13·21 Under conditions where water is readily obtainable the practice has been to prepare the soil-emulsion mix at a relatively high moisture content (above the plastic limit of the soil) to obtain good mixing. This is not always possible in the British Isles, owing to the difficulty of subsequently drying out the mixture for the purpose of compaction.

13·22 A process has been developed for use in this country in which a bituminous emulsion has been used in conjunction with cement[3]. The emulsion, which was specially developed for the purpose, will remain stable for a short time when mixed with fine-grained soils thus promoting good dispersion of the bitumen phase throughout the soil. The cement, which is added subsequently, has three functions, viz: it causes the emulsion to break, absorbs some of the resulting excess free moisture in the soil by hydration and gives added strength to the processed and compacted soil when it has hardened. The proportions of the admixtures generally required in practice are 5 to 7½ per cent of emulsion and 3 to 5 per cent of cement. The material finally produced has properties intermediate between those of soil-cement and true soil-bitumen in that it possesses some rigidity and is also fairly waterproof. The method has also been applied to low-grade aggregates other than soil, such as hoggin, clinker and ash, for use in parking areas and school playgrounds, and it has also been successfully employed in resurfacing old water-bound macadam roads[3].

Bituminous Materials as Waterproofing Agents

13·23 When bituminous materials are mixed with soil the mixture does not absorb water so readily as untreated soil even when relatively small quantities

(2 to 4 per cent) of bitumen are used. This waterproofing effect is believed to be caused by the formation of a film of bituminous material on the surface of the water in the interstices of the soil. The entry of further water into the soil is then hindered by the resistance of the bituminous film to displacement. It has been found that the presence of some wax in the bitumen results in the production of very rigid films, and bitumens having a high natural wax content are therefore particularly suitable for this form of stabilization. Alternatively a suitable amount of wax may be added to a bituminous material deficient in this constituent: a product of this type is available commercially for soil stabilization, under the name of S.S.O./c (" Soil Stabilizing Oil "), which consists of a fuel oil containing an appropriate amount of wax. A proportion of 1 to 2 per cent of hydrated lime is used with this material to flocculate the clay particles in the soil and so reduce the tendency of the soil to absorb water.

13·24 S.S.O. has been used with some success on an experimental scale both in this country and in the colonies; experience in the laboratory suggests that it is a suitable material with which to waterproof cohesive soils at a low moisture content.

13·25 As noted previously in connexion with cut-back bitumen the mixing process has a considerable influence on the performance of bituminous waterproofing agents. Fig. 13·4 shows the variation of water absorption with mixing time for specimens of silty clay containing 3 per cent of a waterproofing oil and indicates that excessive mixing can reduce the waterproofing. Endersby[4] suggests that the waterproofing effect is due to the coating of aggregates of soil particles by fairly thick films of bitumen. He infers that the adverse effect of too much mixing is due to these aggregates breaking down, so that the bitumen is spread over a larger area in thinner films which are not so waterproof.

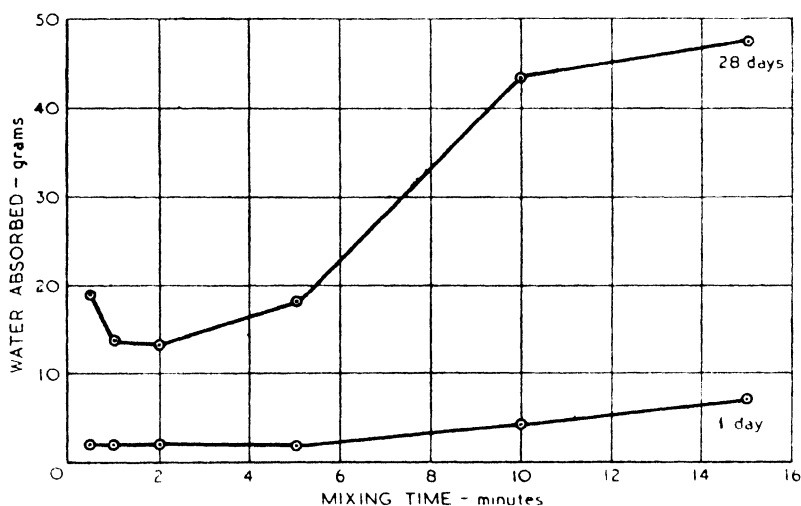



FIG. 13·4 RELATIONSHIP BETWEEN WATER ABSORPTION AND MIXING TIME FOR SPECIMENS OF SILTY CLAY CONTAINING 3 PER CENT WAXED BITUMEN

13·26 In the U.S.A. and Canada waterproofing by bituminous materials is applied to mechanically stable soil bases, i.e. bases made with soils containing gravel, sand and clay in suitable proportions. Typical particle-size distributions of such soils recommended by the Highway Research Board of America[2] are given in Table 13·1.

TABLE 13·1.

TYPICAL PARTICLE-SIZE DISTRIBUTIONS OF SOILS FOR BITUMINOUS STABILIZATION (HIGHWAY RESEARCH BOARD OF AMERICA)

B.S. sieve	Percentage passing		
	1½-in. max. size	1-in. max. size	¾-in. max. size
1½ in.	100	—	—
1 in.	80 —100	100	—
¾ in.	65 — 85	80 —100	100
3/16 in.	40 — 65	80 — 75	80 —100
No. 7	25 — 50	40 — 60	60 — 80
No. 36	15 — 30	20 — 35	30 — 50
No. 100	10 — 20	13 — 23	20 — 35
No. 200	8 — 15	10 — 16	13 — 30

13·27 It is stated that the material passing the No. 36 B.S. sieve should not be less than 40 per cent of the fraction passing the No. 7 B.S. sieve and should have a plasticity index less than 10 per cent, although in certain cases a plasticity index up to 15 per cent may be permitted. In addition the fraction passing the No. 200 B.S. sieve should in no case be more than 60 per cent and preferably not more than 50 per cent of the fraction passing the No. 36 B.S. sieve.

13·28 The bituminous waterproofing agent may be either a tar (RT.4), an emulsion or a cut-back bitumen. The latter should have a viscosity of 3 to 7 sec. (S.T.V.) at 25°C. and be of the RC.1 type. The amounts required vary with the particle-size distribution and the climatic conditions, i.e. 1½-in. and 1-in. maximum size materials require a minimum of 1 per cent of admixture in dry climates and a minimum of 2 per cent when moderate or heavy rainfall may be expected. For the ¾-in. maximum size material the corresponding figures are 2 and 3 per cent respectively.

CONSTRUCTION METHODS

13·29 Construction methods for bituminous stabilization are in general very similar to those used for soil-cement construction (see Chapter 15) and the same plant may often be used. Certain differences in practice exist however, and the more important of these are described below.

13·30 As the optimum moisture content for stability is usually somewhat below that for compaction (Fig. 13·5) and as good mixing is generally considered to be most easily obtained at fairly high moisture contents, it is often found necessary, except with sands, to allow a period for the mix to dry between the mixing process and compaction. To assist in obtaining the highest possible stability the practice is sometimes followed of spreading and compacting sufficient of the mix to give a 2-in. layer leaving the remainder of the material

in a windrow until this layer has dried to a fairly low moisture content. A further 2-in. layer is treated in the same way and if necessary the process is repeated until the required thickness has been built up.

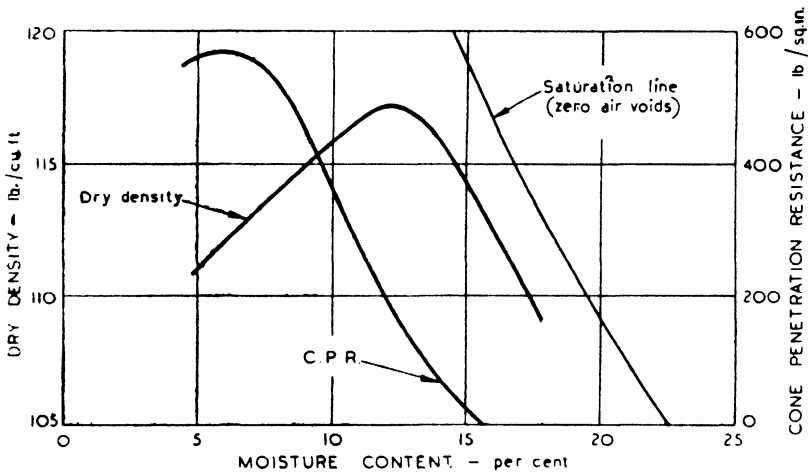



FIG. 13·5 TYPICAL RELATIONSHIP BETWEEN DRY DENSITY, MOISTURE CONTENT AND CONE PENETRATION RESISTANCE OBTAINED DURING A B.S. COMPACTION TEST

13·31 It is usually advisable to apply a surface dressing to a bitumen-stabilized soil but the double application desirable for soil-cement may not be required.

13·32 In the mix-in-place process it is usual to add the bitumen in several passes of the tank-sprayer, each layer being partially mixed in before the next pass, so as to avoid saturating the surface of the soil

13·33 In the bitumen-emulsion/cement process the cement should not be added until after the mixing of the emulsion is completed or else the emulsion may break prematurely. The usual method of adding the cement is by spotting bags at the appropriate intervals and spreading with a shovel, but it is possible that an agricultural lime spreader would do the work equally well for low rates of application.

COSTS

13·34 Current costs of this type of work are not easy to estimate since little bituminous stabilization has been carried out in this country since the recent war. Such figures as are available suggest that the cost will be similar to that for soil-cement construction. On this basis the cost of processing a 6-in. thickness by the mix-in-place process or by a travelling plant would be 2s. to 5s. per sq. yd and by a central plant 5s. to 7s. 6d. per sq. yd (1950 estimate).

LABORATORY TESTING TECHNIQUES

13·35 Testing procedures used in bituminous stabilization may be divided into three groups according to their functions, viz. compaction, stability and waterproofing tests.

Compaction test

13·36 This is carried out with the apparatus used in the B.S. compaction test for unstabilized soil, and the procedure is similar to that used with soil-cement (see Chapter 12). A soil sample is "quartered" into a number of 2-to 2·5-kgm batches, to which different amounts of water are added, and the batches of damp soil are stored in airtight containers for 24 hours or more to allow the moisture to become uniformly distributed. Bituminous stabilizer is then added in a proportion corresponding to the middle of the range to be tested, or to the proportion to be used in subsequent stability or waterproofing tests. Mixing is carried out in an efficient type of laboratory mixer for about 2 min., or for the optimum time for the particular soil, moisture content and mixer that are being used, if this is known. Samples of the various batches are then compacted, and determinations of dry density made at the different moisture contents used. The latter can be determined by the solvent distillation method (see para. 13·68).

13·37 When calculating the results, the weights of both bitumen and water must be subtracted from that of the compacted sample to obtain the dry density. The point corresponding to the peak of the curve relating dry density and moisture content is referred to as the optimum moisture content. Alternatively the percentage of bituminous material present may be added to this figure to give an optimum total fluids content.

Stability Tests

13·38 The three most widely known tests employed to evaluate the stability of a soil-bitumen mixture are the modified Hubbard-Field test, the cone penetrometer test and the Florida bearing value test, the latter test being used only for sands. At the Road Research Laboratory however, the California bearing ratio test (see Chapter 19) has been found to be particularly suitable as a stability test for bitumen treated soils.

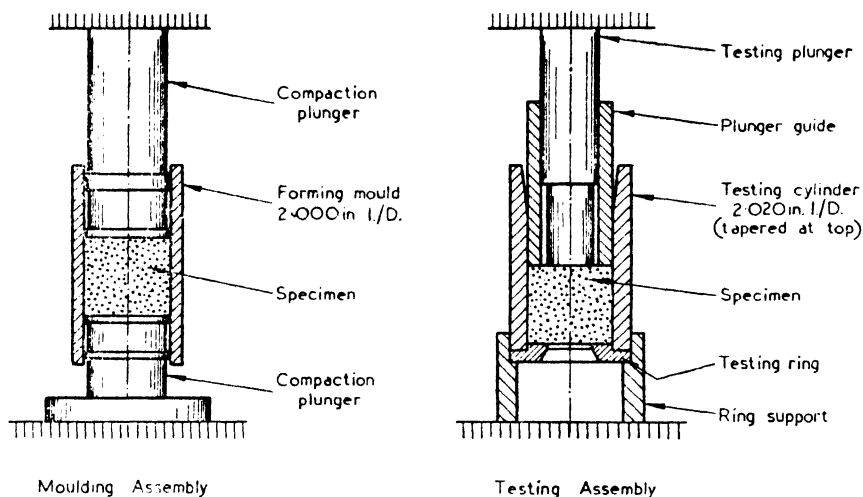



FIG. 13·6 APPARATUS FOR THE MODIFIED HUBBARD-FIELD STABILITY TEST

13·39 MODIFIED HUBBARD-FIELD TEST. This is the most widely used stability test for bituminous stabilization in the U.S.A., and is carried out with the apparatus shown in Fig. 13·6.

13·40 Specimens of the soil-bitumen mixture, 2 in. in diameter and 2 in. high, are prepared in the moulding assembly under a static load of 6,000 lb. After standing the specimens in water to a depth of 1 in. for 7 days their swelling is measured and the specimens are tested in the assembly shown by measuring the load required to force the plunger into the specimen at a rate of 1 in. per min.

13·41 The requirements for satisfactory performance are that neither the swelling nor the water absorption should exceed 2 per cent and the stability should not be less than 400 lb. If the strength is measured without soaking the specimen it should not be less than 1,000 lb., but this procedure is not recommended except as an emergency measure.

13·42 CONE PENETROMETER TEST. In this test, which can be made more rapidly than the modified Hubbard-Field test, a measurement is made of the resistance offered by the soil-bitumen mixture to the penetration of a right-angled cone. The Institute of Petroleum(5) have standardized a procedure whereby the apparatus can be employed to study the variation of stability with moisture content during a B.S. compaction test on a soil-bitumen mixture.

13·43 A typical cone penetrometer for use with thin specimens is shown in Plate 13·1. The 90° cone is mounted on a shaft that can move freely in the vertical direction. This shaft is fitted with a locking nut and a device which enables the height of the cone above the specimen to be adjusted. The shaft also carries a circular platform on which weights may be placed and a dial gauge records the vertical movement of platform and shaft.

13·44 A stabilized soil mixture is prepared at the appropriate bitumen and moisture contents and samples are compacted into the B.S. compaction mould by means of the 5½-lb. rammer. If a compaction test is being carried out simultaneously, the mixture should be compacted in three approximately equal layers in the normal manner. The surface is trimmed flat with a straight-edge and the mould and contents weighed to determine the bulk density. The mould is then covered with a glass plate to prevent evaporation of moisture and allowed to stand for 24 hours before testing, to enable the bituminous material to develop its full effect.

13·45 After this period of curing the glass plate is removed and the penetrometer is placed above the mould and adjusted so that the tip of the cone makes scratch contact approximately at the centre of the surface of the specimen. The scale or dial-gauge is read, the penetrometer shaft is unclamped and the cone gently lowered on to the specimen and a second reading taken after one minute. The difference between the two readings (ρ_1) is the penetration under the load of the cone assembly alone (W_1). A weight sufficient to produce a total penetration of not more than 7 mm. (0·28 in.) is then placed on the platform (a weight of 40 to 50 lb. or 20 to 25 kgm is generally suitable), and the cone is again lowered gently and a reading taken. The difference between this reading and the previous one gives the penetration (ρ_2) under the load of the cone assembly and the additional weight (W_2).

13·46 The cone penetration resistance (C.P.R.) of the soil is then calculated from the formula:—

$$\text{C.P.R.} = \frac{(\sqrt{W_2} - \sqrt{W_1})^2}{\pi(\rho_2 - \rho_1)^2} \text{ (kgm/sq. cm.)}$$

13·47 Determinations of the cone penetration resistance are usually made on a number of specimens at different moisture contents and the relationship between cone resistance and moisture content is established. This is often a hump-shaped curve of the type shown in Fig. 13·5.

13·48 The minimum value of cone penetration resistance for satisfactory stabilization is not known but Jackson[6] has suggested that a resistance equivalent to 20 kgm/sq. cm. is required. Experience in recent work at the Laboratory indicates that a criterion of 80 kgm/sq. cm. is more suitable.

13·49 FLORIDA BEARING VALUE TEST. This test was devised to determine whether a sand is suitable for stabilization with bitumen emulsion or tar. The apparatus (Plate 13·2) consists of a lever-loaded plunger with a circular base 1 sq. in. in area bearing on the surface of a specimen of the sand contained in a mould 3 in. in diameter and 3 in. high. The test is described in detail elsewhere [7] [8] but briefly the procedure is as follows:—10·5 cc. of water are added to 600 gm of air-dried sand. As much of the wet sand as can be pressed in by hand is placed in the mould and compacted by means of a 1,200-lb. load applied to a bearing plate approximately 3 in. in diameter resting on the sand. After compaction the test plunger is placed on the sand surface, and loaded by running lead shot into the bucket until failure occurs and the plunger sinks in. No rate of loading is specified but at the Road Research Laboratory a rate of 10 lb. in 70 sec. has been found satisfactory. The State Road Department of Florida quote a figure of 25 lb./sq. in. as the requirement for the satisfactory performance of sand intended for bituminous treatment.

13·50 MODIFIED FLORIDA BEARING VALUE TEST. A modification of the above procedure has been advocated by McKesson and Mohr[7] for testing bitumen-emulsion/sand mixes constructed in warm climates.

13·51 A particle-size analysis of the sand is first made to determine the percentage by weight of emulsion (W) that will probably be required. This is calculated from the formula:—

$$W = 0{\cdot}0375\,A + 0{\cdot}075\,B + 0{\cdot}375\,C$$

where A = Percentage of sand retained on the No. 7 B.S. sieve.

B = Percentage of sand passing the No. 7 B.S. sieve and retained on the No. 200 B.S. sieve.

C = Percentage of sand passing the No. 200 B.S. sieve after wet sieving.

13·52 When the bearing test is carried out, air-dried sand is mixed with a proportion of water equal to about half the quantity of emulsion to be added; the emulsion is then mixed in together with some additional water if a good mix is difficult to obtain. Extra water added at this stage should be removed after mixing by partial drying in an oven at 60°C. Sufficient sand-emulsion mixture to form a compacted specimen 3·0 ± 0·25 in. high is then rodded into

a mould 4 in. in diameter and compacted under a static load of 25,000 lb. (2,000 lb./sq. in.). The specimen is then placed in an oven at 60°C. for 2 hours after which it is tested, in the apparatus previously described, at a temperature of 60°C. maintained by a constant-temperature bath. The load is applied to the plunger at a rate of approximately 92 lb./sq. in. per min. until failure occurs, which is regarded as taking place when the penetration of the plunger exceeds 0·25 in. or when radial cracks approximately 0·75 in. long appear round the piston. The criterion for successful performance is set at a bearing value of 100 lb./sq. in., and it was found that most sands that have a bearing value of more than 30 lb./sq. in. in the original test when untreated, have bearing values of over 100 lb./sq. in. when mixed with emulsion and tested by the modified method.

Waterproofing Tests

13·53 CAPILLARY WATER ABSORPTION TEST. The capillary water absorption (C.W.A.) test described below was devised at the Road Research Laboratory for testing the efficiency of waterproofing stabilizers and is a development of the method standardized by the Institute of Petroleum[6].

13·54 B.S. compaction tests are made with proportions of stabilizer either in the middle of the range of stabilizer contents to be tested or near each end of the range. This range depends on the stabilizer used, and should be selected to include the value which previous work has shown will give a maximum waterproofing effect. At the Road Research Laboratory stabilizer contents from 1 to 6 per cent by weight of the dry soil are frequently used.

13·55 The results of the compaction test are plotted as a relationship between dry density and total fluids content. With liquid waterproofing agents such as oils, cut-back bitumens, etc., the stabilizer is included with the water in the total fluids content of the soil. A line showing the theoretical relationship between dry density and total fluids content for mixtures containing 10 per cent of air voids is then plotted on the same graph. An illustration of this construction is given in Fig. 13·7 with the compaction curve for a sandy clay soil containing 3 per cent of waxed bitumen.

13·56 Capillary water absorption specimens are then prepared at the dry density and moisture content given by the point of intersection of the 10 per cent air-voids line and the compaction curve. In making up the specimens, a weight of stabilized soil calculated to give the required dry density is placed in a cylindrical mould (Plate 13·3) and the pistons of the mould compressed into position by a hydraulic jack in a suitable frame to form cylinders of a constant volume, 2 in. in diameter and 3 in. high. More recently, it has been found convenient to prepare specimens 2 in. in diameter and 4 in. high, enabling the mould employed in the unconfined compressive strength test for soil-cement (Chapter 12) to be used.

13·57 The specimens are made up with a fairly high air-voids content, since it has been found that most saturated soils absorb water comparatively slowly even in the absence of a waterproofing agent. Consequently, unless there are some air-spaces present in the specimen that can be occupied by incoming water, a false impression may be obtained of the waterproofing efficiency of the stabilizer. A condition corresponding to an air-voids content of 10 per

cent on the B.S. compaction curve has been selected, partly because the dry density obtained is usually numerically very near the maximum obtained in the compaction test, and partly because samples with higher air-voids contents are weak mechanically and are difficult to produce and handle. The corresponding moisture content is in addition near the optimum for stability.

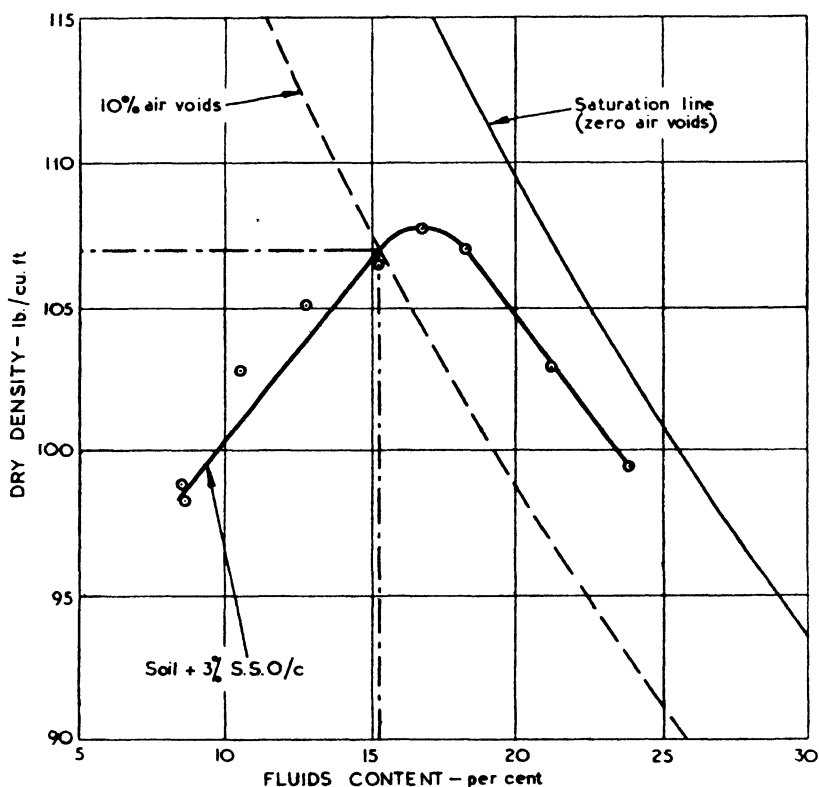



FIG. 13·7 DRY DENSITY/FLUIDS CONTENT RELATIONSHIP (B.S. COMPACTION) FOR A SANDY CLAY CONTAINING 3 PER CENT WAXED BITUMEN

(Required moisture content and dry density for capillary water absorption specimens shown by broken lines)

13·58 A further advantage of making specimens at constant volume and with a fixed amount of air-voids is that it is possible to predict reasonably accurately when the specimens are saturated, if it is assumed that no swelling takes place. Specimens prepared in the manner described contain 15·4 cc. of air, and consequently should be saturated when 15·4 gm of water have been absorbed. Similarly, specimens 4 in. high will require 20·5 gm of water to fill the voids.

13·59 After preparation, the specimens are weighed and measured, to check the bulk density, and completely coated with a layer of paraffin wax to maintain the moisture content at a constant value. The specimens are then stored for

three days at a relatively constant temperature, to allow the stabilizer to develop its full effect. After the storage period, the paraffin wax is scraped off the top and bottom of each specimen to allow water to be absorbed freely.

13·60 The specimens are then placed on suitable carriers, weighed, and placed on a perforated plate in a second metal tank in which water is maintained at a level of 2 mm. above the bottom of the specimen. The weight of water absorbed (in gm) is then determined at intervals of 1, 3, 7, 14 and 28 days by finding the increase in weight of the specimens.

13·61 Fig. 13·8 shows a typical series of water absorption curves for a sandy clay soil containing different quantities of a waxed bitumen, together with the line showing the theoretical amounts of water required to saturate the specimens. Experience at the Road Research Laboratory suggests that the water absorbed after 28 days should not exceed 6 gm for satisfactory stabilization.

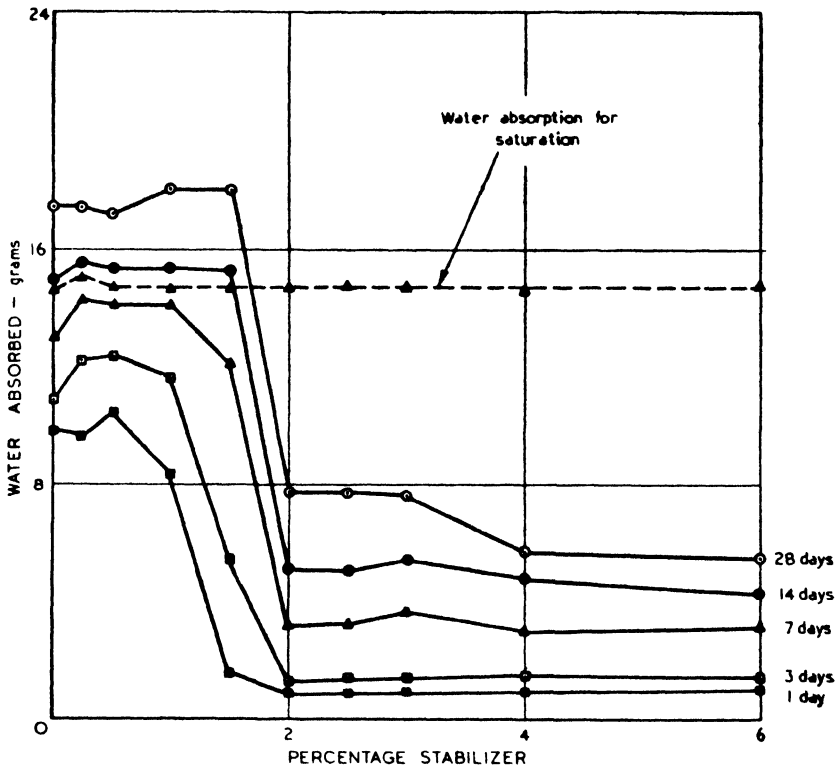



FIG. 13·8 WATER ABSORPTION OF SPECIMENS OF SANDY CLAY CONTAINING VARIOUS PERCENTAGES OF WAXED BITUMEN

13·62 UNCONFINED COMPRESSIVE STRENGTH TEST. This is often carried out when the bitumen emulsion/cement process is to be employed. A range of mixtures of soil, emulsion and cement are tested, the emulsion contents ranging from 1 to 5 and the cement contents ranging from 3 to 15 per cent of the dry soil weight.

13·63 The procedure employed is that described in connexion with soil-cement testing (Chapter 12), except that some specimens are tested after immersion in water, to assess the waterproofing effect of the emulsion. Such specimens are not coated with paraffin wax after moulding.

13·64 A number of specimens are made from a mix, of which half are tested in compression after 24 hours. The remainder are weighed, immersed in water for 7 days, weighed again and also tested in compression. Criteria that have been used in the bitumen-emulsion/cement process are that mixtures to be employed in road construction should have a minimum compressive strength of 150 lb./sq.in. after immersion, while for footpaths or cycle-tracks, a compressive strength of 100 lb./sq.in. is recommended.

FIELD CONTROL TESTS

13·65 On works of any magnitude control tests should be carried out while the mixture is being processed, and the density should be measured after compaction. The tests carried out are:—

(1) Determination of moisture content before and during processing.
(2) Determination of bitumen content after mixing.
(3) Measurement of dry density after compaction.

Determination of Moisture Content

13·66 The moisture content of the soil should be measured at the following times if they are appropriate to the method of construction employed:—

(1) Before processing, to determine how much water has to be added for mixing.
(2) Before compaction (of each layer), to determine whether the soil is at a suitable moisture content for compaction.
(3) Before placing additional stabilized material on to material already compacted, to determine whether the compacted mix has dried sufficiently to develop high stability.

13·67 When determining moisture contents before the stabilizer is added or where an emulsion is used, the normal oven-drying method may be used, but for mixtures containing bituminous stabilizers a method such as that involving solvent distillation described below must be used. In this, it is possible to remove the water and the stabilizer simultaneously and estimate the former separately. Rapid methods of determining the moisture content involving heating to above 110°C. should not be used with an emulsion, but are permissible with untreated soil.

13·68 SOLVENT DISTILLATION METHOD. In this method, which is based on a standard procedure adopted by the Institute of Petroleum[9], a weighed quantity of the mix (50 to 100 gm) is wrapped in a filter paper and placed in a wire gauze basket in the apparatus shown in Plate 13·4. About 100 ml. of water extraction spirit are poured into the distillation vessel, either a conical glass flask or a cylindrical metal pot. The 25-ml. graduated glass receiver attached to the vessel should be fitted with a reflux condenser in the manner shown. The spirit is then boiled gently for 1 to 1½ hours by which time all the moisture in the specimen should have been driven off and collected in the receiver where

its volume can be measured. The wire gauze basket is removed, dried in an oven at 105 to 110°C. and weighed, and the weight of dry soil remaining is determined by difference. Since the weight of the water collected (in gm) is numerically equal to its volume (in cc.) the moisture content can then be calculated by expressing the weight of water recovered as a percentage of the weight of dry soil. The results will not be strictly accurate since the " dry " soil obtained at the end of the extraction will probably contain a small quantity of unextracted bitumen, but the correction is small and for practical purposes can be ignored. If any of the fine material in the soil has passed into the stabilizer solution, it may be estimated either by filtration or by centrifuging the solution; the weight of any such material should be added to that of the soil in the basket.

13·69 When it is not necessary to estimate stabilizer and moisture contents simultaneously, a modification of the above technique may be used. The apparatus is similar in principle, and consists of a 500-ml. round-bottomed flask, a 25-ml. receiver and a condenser, connected by standard ground glass joints. (When coarse- and medium-grained soils of the type described in Chapter 12 are tested, larger metal containers and a different method of joining the components is employed.) A 30- to 200-gm sample of the stabilized soil and 30 to 200 ml. of water-extraction spirit are placed in the flask, and the distillation carried out in the manner described above. It is necessary to know or assume a value for the stabilizer content in calculating the moisture content on a dry soil basis in this case. In both this and the preceding method, the water-extraction spirit can be recovered using the same vessel and condenser connected with a suitable adaptor.

Determination of Bitumen Content

13·70 Two methods are available[10] for determining the bitumen content of a stabilized soil mix. The cold extraction method using methylene chloride is suitable for field control and gives results in 30-40 min. The hot extraction method can be carried out simultaneously with the solvent distillation method for determining the moisture content and uses the same apparatus but takes 4-6 hours to perform. Whichever method is used blank determinations should be made on the untreated soil. A different procedure is required for the determination of tar content[11].

13·71 COLD EXTRACTION METHOD. A 200-gm sample of the soil mix is weighed out and placed in a 500- to 600-ml. metal bottle with sufficient powdered silica gel to absorb the water present (1 part of silica gel to 2 parts of water by weight). Sufficient methylene chloride to give a 2 to 3 per cent solution of the stabilizer is measured to the nearest 0·5 ml. and poured into the bottle. Three steel balls of 1-in. diameter are then added to assist in breaking down the lumps of soil and the bottle is then tightly stoppered with a rubber bung and rotated end over end at 60 r.p.m. for 30 min. on a suitable mechanical shaker (Plate 13·5). The solution is then filtered under pressure from a foot pump through a previously prepared alumina filter candle into a burette. The filter should be converted to an enclosed type by passing a length of glass or metal tubing through a cork or metal ring at the open end and sealing this with a cement paste composed of copper oxide powder (prepared by direct oxidation of metallic copper) and phosphoric acid.

13·72 Sufficient of the solution is run from the burette into a weighed carbon dioxide flask to provide 0·75 to 1·25 gm of the stabilizer. The solvent is then evaporated at a pressure of 55 to 65 cm. of mercury by connecting the flask to a vacuum line fitted with a manometer and reservoir and immersing it in a water bath at 100°C. (Plate 13·6). Should a filter pump be insufficient to give the required vacuum a mechanical pump may be used. This should be connected to the flask through a reservoir containing medium-viscosity lubricating oil, followed by an absorption tower containing activated carbon of 12-20 mesh, to ensure that solvent vapour does not reach the pump. (Most of the solvent should be distilled off at 100°C. before the flask is connected to the pump.)

13·73 When foaming occurs in the last stages of evaporation, the procedure is varied according to the stabilizer used. If an emulsion is used, the pressure is reduced to 16 cm. of mercury in 1½ min. and maintained for 3½ min. If a petroleum oil or cut-back bitumen is used, the pressure is raised to approximately atmospheric pressure and reduced to 46 cm. of mercury at 20 cm./min. and maintained for 3½ min. The flask is then removed from the bath, dried on the outside, and a gentle current of air blown through to remove the last traces of methylene chloride. After cooling in a desiccator the flask is reweighed and the weight of binder remaining is determined. The bitumen content of the original sample, based on the dry weight of the soil, can then be calculated

13·74 It is also necessary to determine the moisture content of the mixture simultaneously and this can be done by the second method described above.

13·75 HOT EXTRACTION METHOD. The apparatus required for this method is the same as that described above for the first solvent distillation method of determining moisture content.

13·76 The procedure is the same as that for determining the moisture content up to the point where the volume of water is measured. As much as possible of the spirit is then decanted off and replaced by 100 ml. of a mixture of 95 parts by volume of trichlorethylene with 5 parts by volume of methyl alcohol. The change of solvent should not take place until at least 1½ hours has elapsed after the beginning of the determination of the moisture content. Refluxing should be continued until all the stabilizer is removed from the sample (about 1½ hours for an emulsion and up to 4 hours for a stabilizing oil). The basket is then dried in an oven at 135 to 140°C., cooled in a desiccator and weighed. If any of the fine material of the soil has passed into the stabilizer solution it may be estimated either by filtration or by centrifuging the solution; the weight of any such material should be added to that of the soil in the basket.

13·77 If only a determination of bitumen content is required, the mixture of trichlorethylene and methyl alcohol should be used throughout. The reflux condenser is also fitted directly through the stopper of the flask or metal pot and the water receiver omitted. The times of extraction are similar to those for the method using two solvents but it is possible to accelerate the extraction by decanting off the first batch of liquid used and adding fresh solvent after 3 hours.

Determination of Dry Density

13·78 The dry density of the compacted stabilized soil can be determined by either the sand-replacement or the core-cutter method described in Chapter 9.

APPENDIX TO CHAPTER 13

American classification and corresponding viscosity of Bitumens and Tars

BITUMENS

American classification	Approximate viscosity (Standard tar viscometer)
RC.0, MC.0, SC.0	½-1 sec. at 25°C.
RC.1, MC.1, SC.1	3-7½ sec. at 25°C.
RC.2, MC.2, SC.2	11-31 sec. at 25°C.
RC.3, MC.3, SC.3	50-140 sec. at 25°C.
RC.4, MC.4, SC.4	200-800 sec. at 25°C.
RC.5, MC.5, SC.5	110-330 sec. at 40°C.

RC. bitumens are rapid-curing
MC. „ „ medium-curing
SC. „ „ slow-curing

TARS

American classification	Approximate viscosity (S.T.V. or E.V.T.)
R.T. 1.	None
R.T. 2.	None
R.T. 3	None
R.T. 4.	1-2 sec. at 30°C. (S.T.V.)
R.T. 5.	2-4 sec. at 30°C. (S.T.V.)
R.T. 6.	4-8 sec. at 30°C. (S.T.V.)
R.T. 7.	11-35 sec. at 30°C. (S.T.V.), 25°C. (E.V.T.)
R.T. 8.	35-70 sec. at 30°C. (S.T.V.), 30°C. (E.V.T.)
R.T. 9.	70-240 sec. at 30°C. (S.T.V.), 36°C. (E.V.T.)
R.T. 10.	45-110 sec. at 40°C. (S.T.V.), 42°C. (E.V.T.)
R.T. 11.	110-180 sec. at 40°C. (S.T.V.), 46°C. (E.V.T.)
R.T. 12.	60-150 sec. at 50°C. (S.T.V.), 54°C. (E.V.T.)

SUMMARY

13·79 Bituminous stabilization, in which asphaltic bitumens, tars or bituminous emulsions are added to soil, is used in the construction of surfacings and bases for low-cost roads. The bituminous material may either increase the cohesion of the soil by binding the particles together, or it may act as a waterproofing agent and maintain the cohesion owing to thin films of water forming between the particles. The amount of bituminous material required depends upon which of these two effects is desired and upon the soil type. Climatic conditions also influence the concentration that can be employed since they affect the amount of fluid, i.e. moisture, that is already present in the soil. This chapter reviews the processes of soil stabilization that are employed in various parts of the world and describes the characteristics of the materials that are produced. Notes are also included on the construction methods employed and on the costs.

13·80 Brief descriptions are given of the techniques required in the preliminary laboratory testing of soil-bitumen mixtures. These include tests for the stability of the mixture, such as the modified Hubbard-Field test, the cone penetrometer test and modified Florida bearing value test. A description is also given of the procedure developed at the Road Research Laboratory for the capillary water absorption test for determining the waterproofing efficiency of bituminous additives.

13·81 It is desirable to control the composition and compaction of the processed soil and descriptions are therefore included of the field methods by which the moisture and bitumen contents of a soil mixture can be determined and reference is also made to the British Standard methods for determining its dry density after compaction.

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CHAPTER 14

STABILIZATION OF SOIL WITH RESINOUS MATERIALS

INTRODUCTION

14·1 The advent of resinous materials in the field of soil stabilization is comparatively recent and, although showing promise, they are still only in the development stage. Insufficient data are as yet available from field experiments to show whether the processes concerned are completely satisfactory: the information so far obtained suggests that they are much more dependent upon soil conditions than the other methods of stabilization.

14·2 The areas in which resinous waterproofing methods can be applied are restricted by the moisture conditions of the soil. In the British Isles, where cohesive soils have a relatively high moisture content for long periods of the year, the addition of waterproofing agents may serve only a limited purpose. In colonial territories, however, in which long dry seasons alternate with periods of heavy rainfall, it may be desirable to waterproof cohesive soils when they are relatively dry.

14·3 The resinous materials to be considered are all natural products or obtained by processing natural resins. Some synthetic resins of the type used in plastics have been proposed for use in soil stabilization, e.g. aniline-furfural by Winterkorn[1] and phenol-formaldehyde by Blott[2], but the application of these materials has not been taken farther than the laboratory stage. These synthetic stabilizing agents have the advantage that they not only waterproof the soil, but also increase its strength.

RESINOUS SOIL-STABILIZING AGENTS

14·4 The resinous materials available commercially for soil stabilization are " Vinsol "-resin, and rosin or derivatives of rosin.

14·5 " Vinsol "-resin is a dark brown solid material, melting at about 110 to 115°C. and having a specific gravity of about 1·25. It is manufactured from the resinous residues obtained after the distillation of pine tree stumps for turpentine. These residues are extracted with benzene or toluene which is then removed, leaving a mixture of " Vinsol "-resin and rosin. Subsequent extraction of this mixture with a petroleum solvent removes most of the rosin, leaving the " Vinsol," which may then contain up to 5 per cent of unseparated rosin. The production process has been described by Hall[3], and the use of " Vinsol " for soil waterproofing by Miller[4]. The material is available commercially for this purpose in the form of a very fine powder.

14·6 Rosin is obtained during the steam distillation of oleo-resins from pine trees during turpentine extractions. The commercial product melts in the range 75 to 100°C., is soluble in many organic solvents, and can be dissolved

in the aqueous solutions of the alkali hydroxides. Winterkorn and McAlpin[5] have studied the soil-stabilizing properties of these alkali rosinates, and also of the insoluble rosinates that can be precipitated in the soil by the action of salts of bi- and trivalent metals on solutions of the alkaline rosinates. A process on this basis has been developed in this country using sodium rosinate, usually in conjunction with an aluminium salt[6]. Winterkorn, McAlpin and Mainfort[7] have studied the stabilizing properties of a rosin-alkali complex in which only 25 per cent of the rosin is neutralized with sodium hydroxide. A material of this type has been produced in the U.S.A. under the name of " 321 "-resin, for use as a soil-stabilizing agent.

14·7 At the Road Research Laboratory a study has been made of the waterproofing properties of several tropical resins, including Copal and Damar resins from the Dutch East Indies and the Belgian Congo, and resins from Ceylon, Nigeria and British Guiana. The only one of these materials that was found to have any soil-waterproofing properties was a Manila Copal resin.

CHARACTERISTICS OF RESIN-TREATED SOIL

14·8 Laboratory investigations at the Road Research Laboratory have indicated certain characteristics of resin-treated soils that are of interest to engineers, and these are described briefly below.

Water Absorption

14·9 The main object of incorporating resins into soil is to reduce the water absorption. This can be accomplished for suitable soils with very small percentages of resin (1 to 3 per cent) as shown by Fig. 14·1, in which the water absorption of cylindrical specimens of a sandy clay soil (see " capillary water absorption test ") is plotted against the percentages by weight of resin incorporated in the specimens. The dotted line indicates the amounts of water that would be required to saturate each specimen. After seven days the samples containing no resin were completely saturated, whereas those containing 1 per cent of resin had absorbed about 1·5 gm of water and were therefore only 10 per cent saturated.

14·10 The waterproofing effect is of the same order under the experimental conditions employed, between resin contents of 0·75 and 2·0 per cent. When the resin content is increased to 3 per cent, however, the water absorption increases. This inferior performance at higher resin contents has been noted in a number of cases and suggests that the stabilizer content is critical to within 1 per cent.

Compaction

14·11 When resinous materials are added to soil and a B.S. compaction test is carried out immediately afterwards, the results obtained are similar to those obtained with the same soil containing no resin (see Fig. 14·2). However, if the soil-resin mix is allowed to stay in the loose, uncompacted state for some days prior to the compaction test, the maximum dry density is reduced and the optimum moisture content increased. This suggests that, although soil-resin mixtures can be stockpiled for maintenance work, it is usually desirable to compact them as soon as possible after mixing.
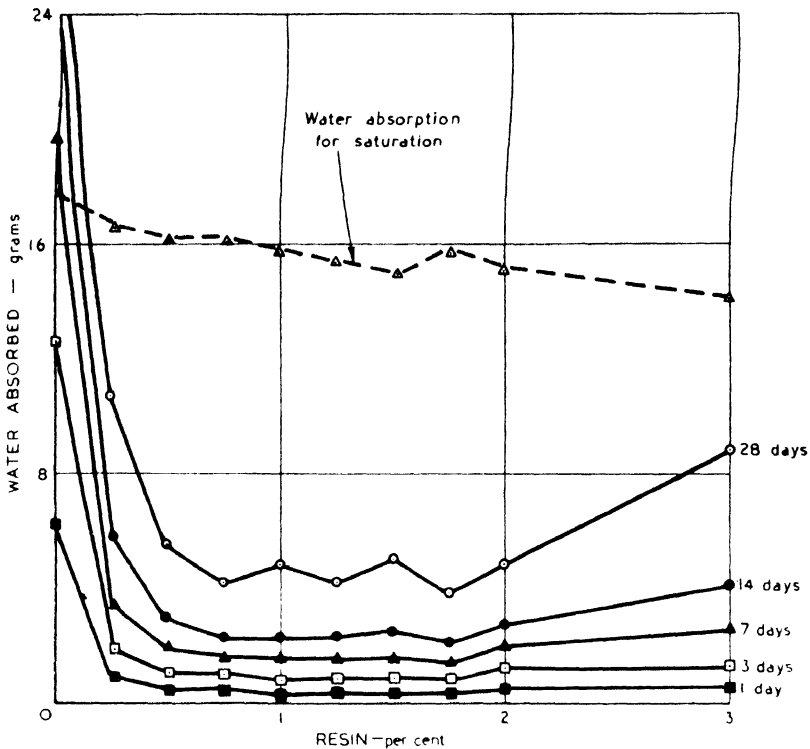
FIG. 14·1 WATER ABSORPTION OF SPECIMENS OF SANDY CLAY CONTAINING VARIOUS PERCENTAGES OF "VINSOL"-RESIN

Stability

14·12 Laboratory experiments, in which soil-resin mixtures were subjected to direct shear tests, have shown that neither the angle of shearing resistance nor the apparent cohesion of a soil is appreciably affected by the presence of resin.

Soil Reaction

14·13 The effect of the soil pH (see Chapter 2) on the waterproofing effect of a resin is shown in Fig. 14·3. This gives the amounts of water absorbed after different periods by capillary water absorption specimens of an acid sandy clay soil containing 1 per cent of gum rosin, when the pH of the soil has been adjusted to different values by the addition of lime. The rosin was found to exert a waterproofing effect in all cases where the soil reaction was acid, but where the soil was alkaline (pH in excess of 7) there was a definite increase in water absorption. It is, therefore, concluded that natural resins of this type are only suitable for use in acid soils. It is understood that similar results have been obtained in field experiments in the U.S.A.
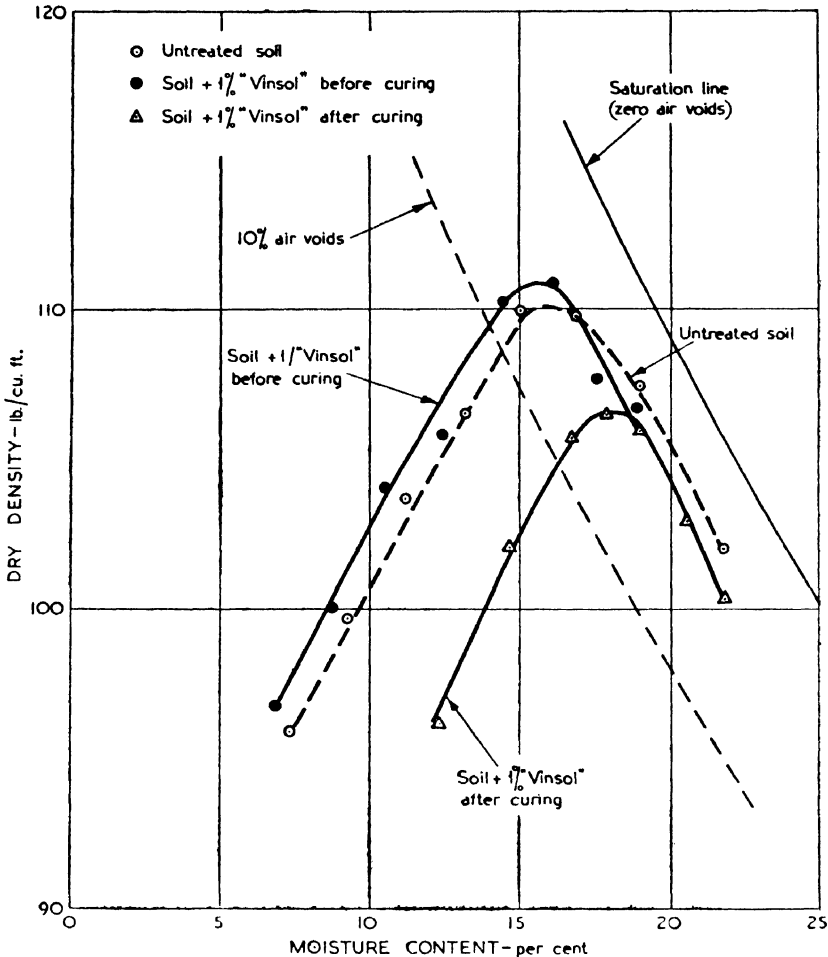
FIG. 14·2 DRY DENSITY/MOISTURE CONTENT RELATIONSHIP (B.S. COMPACTION) FOR SANDY CLAY CONTAINING 1 PER CENT "VINSOL"-RESIN BEFORE AND AFTER CURING

Alkaline Soils

14·14 Although limitations are imposed on the use of ordinary resins by the pH effect, laboratory tests have shown that some waterproofing effect can be obtained with chalky soils using emulsions. Fig. 14·4 shows the effect of adding proportions of a tall oil emulsion on the water absorption of a sandy clay soil containing 4 per cent of chalk. Tall oil is a resinous by-product obtained in the manufacture of certain types of paper pulp. At relatively high concentrations, of the order of 6 per cent, some waterproofing is obtained, although it is not as good as that obtained with powdered natural resins in acid soils.

Effect of Frost

14·15 Laboratory experiments by Winn and Rutledge[8] have shown that treatment with "Vinsol"-resin reduces the susceptibility of a soil to damage

by frost. These experiments have been confirmed at the Road Research Laboratory, in tests in which specimens of a sandy clay soil prepared in the same manner as in the capillary water absorption test (see Chapter 13) were subjected to a temperature gradient while their lower surfaces were in contact with water. Plate 14·1 shows two typical specimens from these experiments after 24 hours' freezing. The specimen on the left, which was made with untreated soil, has heaved about half an inch and the characteristic ice-lens formation is clearly visible. The specimen on the right, which contained 1 per cent of "Vinsol," has not heaved under similar conditions and shows no signs of damage.
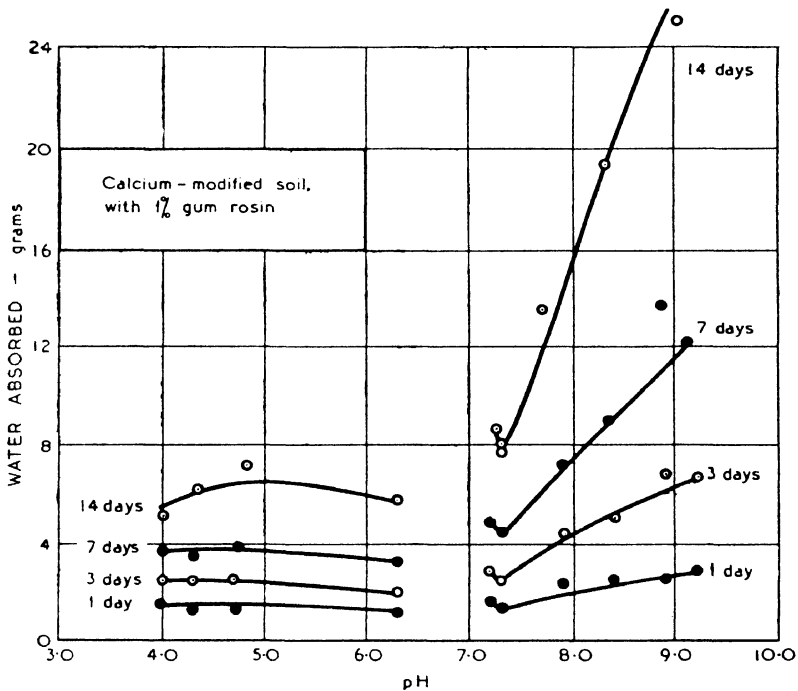
FIG. 14·3 RELATIONSHIP BETWEEN WATER ABSORPTION AND HYDROGEN ION CONCENTRATION (pH) FOR A SANDY SOIL TREATED WITH 1 PER CENT GUM ROSIN

Microbial Attack

14·16 In a microbiological investigation undertaken at the request of the Road Research Laboratory, Jones[9] has shown that both "Vinsol"-resin and rosin are attacked by bacteria and fungi. Under laboratory conditions this attack can be so severe that the waterproofing action of the resin is completely destroyed. It was noted, however, that the bacteria concerned are aerobic, and that the attack can be lessened if the air content of the soil is reduced by compaction. Suitable antiseptics have been studied on a laboratory scale, and a halogenated organic compound, sodium pentachlorophenate, appears to be effective. The general findings of this investigation suggest that
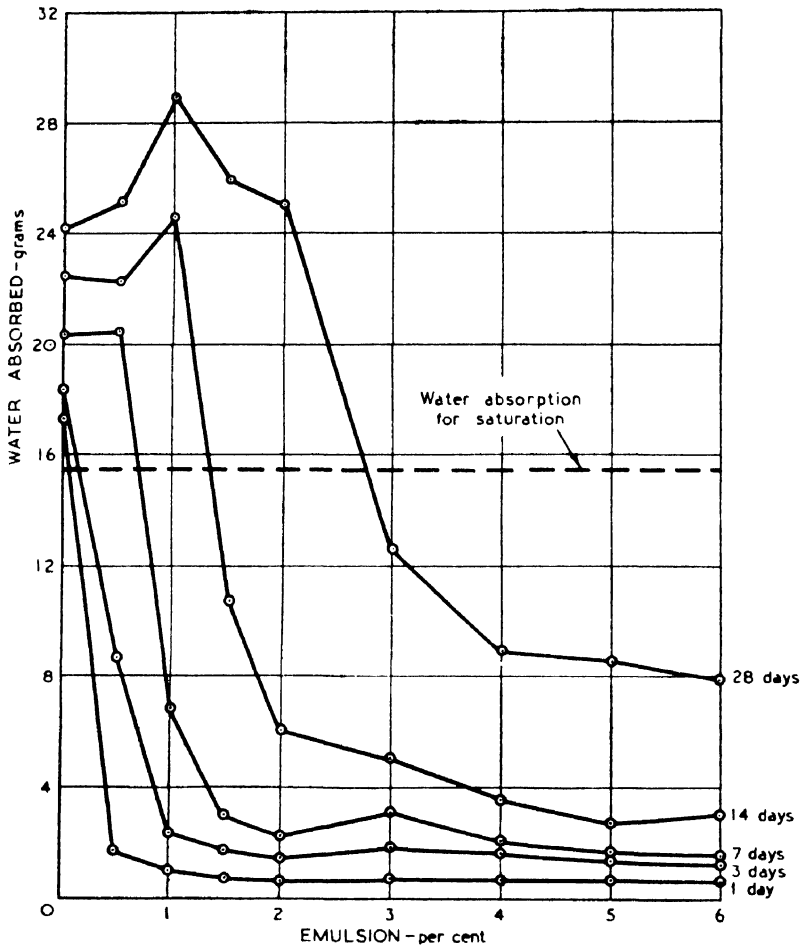
FIG. 14·4 VARIATION OF WATER ABSORPTION OF A SANDY CLAY CONTAINING 4 PER CENT CHALK WHEN TREATED WITH A RESINOUS EMULSION

the waterproofing effect of resinous materials may not be permanent over a number of years, and that their durability may be less under tropical than under temperate climatic conditions. However, under certain conditions such as those existing in soil-cement mixtures, bacterial activity is practically eliminated, and it may be that in future, resinous materials will find a more satisfactory permanent application as additives for cement to be used in soil-cement stabilization.

CONSTRUCTION METHODS

14·17 Field work with resinous soil stabilization has hitherto been on an experimental scale, and it is therefore not possible to indicate in detail the field procedure required. It would appear, however, that when dealing with

powdered resins the methods used in soil-cement stabilization are applicable, while when the resinous material is in the liquid form, as for example sodium rosinate solution, the techniques employed in bituminous stabilization can be used

14·18 Webb[10] has described the construction of an experimental length of resin-treated soil road in this country by mix-in-place methods. Plate 14·2 shows the methods employed at this site for distributing a powdered stabilizer (" Vinsol"-resin) and a fluid stabilizer (sodium rosinate). Owing to the low concentrations required, care is needed in the application of powdered material to ensure as even a distribution as possible.

COSTS

14·19 Since the field work carried out in this country with resinous stabilization has been of an experimental nature, accurate figures for the costs cannot be given. They would probably be similar to those for soil-cement, since the plant, processing and labour items would be similar, and the stabilizer cost would also be of the same order, since 1 per cent of " Vinsol "-resin at £38 per ton would be roughly equivalent to 10 per cent of cement at £3/10/0 per ton. There would, however, be a small saving in the cost of handling and distributing the smaller quantities of stabilizer involved.

LABORATORY TESTING TECHNIQUES

14·20 The tests employed in evaluating resinous soil stabilizing agents are (1) the B.S. compaction test, and (2) the capillary water absorption test.

14·21 The procedure in the compaction test is substantially the same as that described for soil-cement in Chapter 12. Since a reduced dry density is obtained if the soil-resin mix is " cured " before compaction, " curing " is only carried out in the laboratory when it is known that an appreciable time is to elapse between mixing and compaction in the field. Moisture contents are determined by normal oven-drying at 105 to 110°C., and in calculating the dry density of samples the resinous material is included in the total weight of soil. The capillary water absorption test is carried out in the manner described in Chapter 13.

SUMMARY

14·22 The stability of cohesive soils in road bases and foundations can be maintained under adverse moisture conditions by the addition of waterproofing agents to the soil. It has been found that very small quantities (1 to 2 per cent) of certain natural resinous materials have a waterproofing action, and this has been studied in the laboratory. In addition to reducing the water absorption, it has been found that resins also affect the compaction properties of soil, and reduce its liability to damage by frost. Resins are more successful with acid than with alkaline soils but are subject to microbial attack.

14·23 Brief references are made to the laboratory tests required to evaluate resinous waterproofing agents, to the methods by which they are applied in the field and to the cost of construction.

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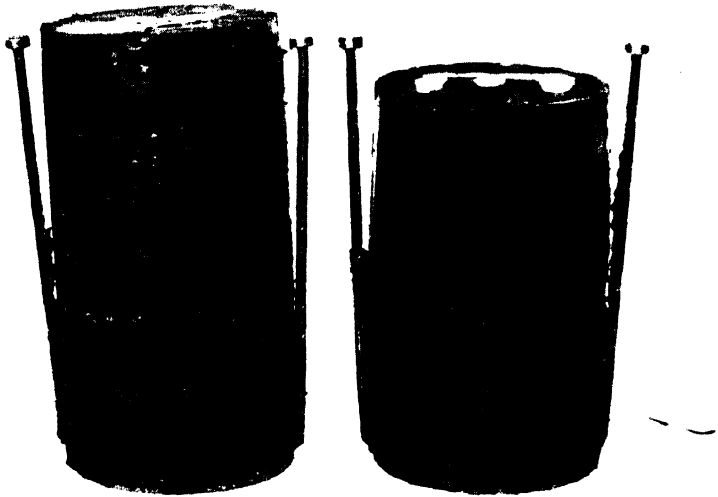

SOIL SPECIMENS AFTER BEING SUBJECTED TO A TEMPERATURE GRADIENT FREEZING TEST

The specimen on the left was made with untreated soil, whereas that on the right was made with soil containing 1 per cent of "Vinsol"-resin

PLATE 14·1

(A) SPADING POWDERED RESIN PRIOR TO MIX-IN-PLACE STABILIZATION

(B) APPLICATION OF A WATER-SOLUBLE RESINOUS MATERIAL PRIOR TO MIXING

PLATE 14·2

CHAPTER 15

CONSTRUCTIONAL METHODS IN SOIL STABILIZATION

INTRODUCTION

15·1 This chapter describes the methods of construction and types of plant that have been developed for stabilized soil roads. The three principal methods of construction are:—

(1) Mix-in-place
(2) Travelling plant
(3) Stationary plant.

MIX-IN-PLACE METHOD

15·2 In this form of construction a " train " of machines is run over the soil to be processed. Rippers, cultivators (Plate 15·1A) or rotary tillers (Plate 15·1B) are first used to break up the soil, while ploughs (Plate 15·2A) or scarifiers ensure that the soil is loosened to a uniform specified depth. Water is then added to the loose soil from a sprayer (Plate 15·2B) to bring it to a suitable moisture content for processing. The stabilizer is then added, either by spray-wagon, in the case of a fluid stabilizer, or by hand (Plate 15·3A) or preferably from a bulk spreader (Plate 15·3B) when the stabilizer is in the form of a powder. Further passes of the rotary tillers or of special soil mixers (Plate 15·4A) mix the stabilizer into the soil. In the past ordinary agricultural equipment, such as harrows, etc., have been used successfully in the mixing process, but now rotary soil mixers are usually considered essential and nearly always used. Shaping of the loose mixed material (Plate 15·4B) is followed by compaction, usually with a suitable roller but sometimes with vibrators.

15·3 In the mix-in-place method no forms are used to define the edge of construction. It is more economic to build the road slightly wider than necessary, and cut back to the desired width when the stabilized construction is completed. Normally, however, the excess stabilized width is left in place and used as a foundation for kerbs. A daily output of 2,000 to 8,000 sq. yd of a 6-in. compacted layer may be expected using mix-in-place technique. The procedure is described below.

Preparation of the Subgrade

15·4 The site is levelled to the required formation and cleared of stumps. boulders and debris for a depth of about 12 in. Level pegs are then driven in to check the formation levels, the depth of treatment and the setting out of the work. They are normally placed on the shoulders at 50- to 100-ft intervals, 4 to 5 ft from the proposed edges of treatment. In the preparation of the subgrade, the topsoil is normally removed, although in some instances topsoils have been stabilized.

Pulverization of the Soil

15·5 This comprises:—

(1) Scarifying to the required depth of treatment.
(2) Pulverizing the scarified soil until a fine tilth is produced suitable for mixing in the stabilizer.

The depth of pulverization must be carefully controlled as processing to an excessive depth reduces the stabilizer content in the mixed material while too shallow processing produces a stabilized layer that is too thin. The depth to which the soil is scarified is normally slightly less than the final thickness of the compacted layer of stabilized soil. The actual allowance to be made can be determined by comparing the bulk density of the subgrade with the designed bulk density to be aimed at in the stabilized construction.

15·6 Suitable plant for cutting up the soil to the required depth is a plough, or robust tiller with a positive depth control. Rippers and cultivators, when used on their own, tend to leave ruts in the subgrade. The plough should always be used to turn the soil toward the centre of the road; this leaves a vertical face of soil at the shoulders and prevents processing being carried outside the limits of the road.

15·7 Rotary tillers are used for pulverization, but disc harrows may be a useful addition with some soils. When pulverization is completed, about 80 per cent or more of the soil, exclusive of stones, should pass a $\frac{3}{16}$-in. B.S. sieve. The loose surface is then shaped with a grader to give an even distribution of loose soil along the length and width of the road.

Application of the Stabilizer

15·8 With solid stabilizers such as cement, the bags are spotted at suitable points on the loose soil, preferably from lorries, and then split open and emptied by hand. Spreading may be accomplished with a spike harrow, a blade grader or hand rakes. On large jobs cement is sometimes spread from bulk cement trucks, containing as much as 25 tons of cement. Liquid stabilizers are sprayed on to the soil from pressure distributors.

Mixing

15·9 At this stage mixing is begun with rotary tillers or special soil mixers. " Dry " mixing is usually done in two or three passes of the machines. It is normally followed by " wet " mixing, when mixing takes place concurrently with the application of water. This is continued until the mixture has a uniform colour; this criterion has to be used as there is no rapid means for assessing the uniformity of mixing.

15·10 Ploughs are not normally used in this part of the process but, if they are used at all, care should be taken to see that the stabilizer is well mixed in beforehand and that " dry " mixing is complete. Otherwise, especially with powdered stabilizers, the plough buries most of the stabilizer at the bottom of the layer to be treated.

Addition of Water

15·11 Even in the climatic conditions experienced in this country, the amounts of water required to be added to the soil are sometimes quite considerable, and

efficient and even distribution is essential. To avoid interruptions two water distributors should be used, one spraying while the other is being filled. These are followed by the mixing machines and processing is continued until sufficient water has been sprayed and the mixture is uniformly damp, an operation which should not last longer than 3 hours. On no account should the water distributor be allowed to stop while spraying, or a wet area, which is difficult to eradicate, will result.

Grading

15·12 Grading is carried out either by towed graders (Plate 15·4B) or by auto-patrols, forms seldom, if ever, being used in mix-in-place construction. In stabilization work the road should not be allowed to get unduly out of shape at any time during construction. Grading should be done at the following stages of the work:—

(1) In the preparation of the subgrade.

(2) When pulverization is complete.

(3 Continuously during mixing. This is to keep the levels as near correct as possible in order to reduce the time taken in the final grading of the loose material.

(4) When mixing is complete. This should be done as rapidly as possible before evaporation losses cause a need for further wet mixing.

(5) Final shaping of the compacted road. This should be avoided altogether if possible, since any grading removes stabilized material. The rollers should leave the road shaped adequately, but if further grading is necessary, then care should be taken to see that the depth of the treated layer is not reduced by an excessive amount.

15·13 Because grading of stabilized soil roads has to be done on loose material and in a relatively short time, it is often found convenient to design roads with a cross-fall instead of a camber in order to facilitate the grading operation.

Compaction

15·14 When mixing is complete and the soil graded to the required section, compaction is begun. The same principles of compaction as described in Chapter 9 are usually followed. Particular care must be taken in the final stages of compaction of soil-cement, however, in order to avoid overstressing the processed soil by the use of a roller that is too heavy, or by rolling for too long. This might reduce the strength of the soil-cement by cracking it or breaking the initial structure formed by the cement as it hydrates.

15·15 Surface finish is another factor which is often of more importance in stabilized soil construction than in subgrade compaction. Final rolling should, therefore, be done with a smooth-wheel roller. The main compaction of gravelly or sandy soils is most suitably done by means of either a pneumatic-tyred or a smooth-wheel roller. For sandy clay soils a pneumatic-tyred roller (Plate 15·5A) is usually the most suitable type of roller. Besides its satisfactory compaction characteristics (see Chapter 9) it is of particular practical use in soil stabilization where it is desirable to reduce the amount of grading necessary as much as possible. This roller tends to avoid the pushing of the soil, and

consequent waviness, which is often produced when a smooth-wheel roller is put directly on to a loose layer of soil. The use of a sheepsfoot roller is not favoured, but if it is used it should be followed by a spike-harrow or nail-drag to remove the marks of the feet. With clean sands, especially uniform sands such as dune sands, vibration is the best method of compaction. In soil-cement work it is desirable to complete compaction within 2 hours, i.e., not more than 5 hours after the beginning of "wet" mixing. Plate 15·5B shows a compacted, but unsurfaced, stabilized soil road laid in a housing estate.

Joint with old work

15·16 Care is needed to ensure that a sound joint is made between new construction and that laid the previous day. Fig. 15·1 illustrates how a soil-cement base can be cut back to sound material and protected, and also how to ensure good mixing of material laid subsequently.

15·17 Small areas adjacent to the main construction, such as lay-byes and the curved areas at road junctions, are best treated separately with a small hand-operated rotary tiller for mixing. Such areas should be joined, in a similar way to that described above, to the surrounding construction previously carried out, using a vertical butt joint.

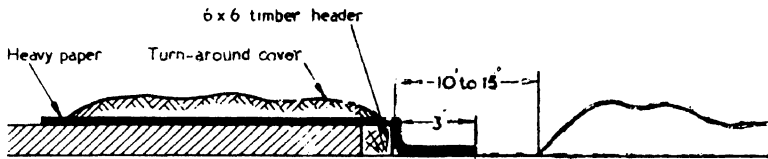
Curing

15·18 Many stabilizing agents require a period of curing after mixing, before they become fully effective. This is particularly necessary with soil-cement, which should be protected for a period of 7 days by covering with a layer of moist soil or straw. Alternatively, the surface should be kept damp by frequent applications of a light spray of water. A bituminous priming coat has also been used as a curing agent by applying it soon after compaction. There are dangers with this method, however, and in some instances a slight surface disintegration has resulted, possibly from interference with the hydration of the cement.

Surfacing

15·19 Before a surfacing is applied, the stabilized soil should be sprayed with a bituminous priming coat, having a viscosity of about 5 to 20 sec. at 30°C. in the standard tar viscometer or its equivalent, depending on the texture of the compacted surface. The priming coat, which provides a key for the surfacing, should be a bitumen if the surfacing contains a bitumen binder, while if the latter contains a tar binder the priming coat should also be a tar. A more detailed discussion of surfacings for soil roads will be found in Chapter 10.

15·20 There are, in general, two forms of practice with regard to the time at which the surfacing is laid. One method is to lay the surfacing soon after the end of the curing period in order to afford maximum protection to the stabilized layer; the other method is to allow traffic on the road for some period, say up to a year, so that any faults can develop and be corrected before the surfacing is laid. The choice of procedure should be related to the soil type and traffic conditions; very often a compromise is made by laying a surface dressing immediately and a thicker surfacing at a later date. It is not felt, however, that a surface dressing is a satisfactory permanent surfacing for stabilized soil, except on very lightly trafficked roads, and in general some form of bituminous carpet is desirable.

1. The completed section is cut back and the pulverized soil of the next section pushed away from the joint

2. A timber header is laid against the edge of the completed section, which is protected by a layer of building paper and a turn-around cover of soil (or planking etc.)

3. The pulverized soil is spread back to the joint, the moisture content of the new section brought to the required value and the cement laid

4. The soil and cement are mixed thoroughly and water added if necessary

5. The freshly processed soil-cement is pushed away from the joint, the paper cut back and the header removed

6. The fresh soil-cement is spread back to the joint and the paper used to separate it from the turn-around cover

7. The new section is compacted and left a little proud of the old work at the joint. The paper and turn-around cover are removed and the raised portion is cut off later

FIG. 15·1 DIAGRAMMATIC SKETCHES ILLUSTRATING THE CONSTRUCTION OF A JOINT BETWEEN TWO SECTIONS OF WORK

Plant used in Mix-in-place Construction

15·21 The effectiveness of mix-in-place construction depends considerably upon the type of plant used in the mixing process. Types of plant available include:—

The Rotary Hoe

15·22 This is a small (4 ft wide) but robust machine with a moderate-speed bladed rotor, driven through a power take-off from a tractor (see 15·1B). Though designed for agricultural work, it has been used in a number of instances for soil stabilization in this country and has been particularly successful in processing coarse-grained soils containing a relatively high percentage of stones. Hand rotary tillers are also manufactured. These are suitable for small areas.

The Seaman Pulvimixer

15·23 This machine is 6 ft wide and has a high speed rotor driven directly from a separate power unit (see Plate 15·4A). The rotor carries a large number of tines and can be lowered into the mixing position by means of a hydraulic ram. One important feature of the machine is the low hood over the rotor which acts as a good screed over the loose soil. The machine has been used extensively in this country and operates particularly well on fine-grained soils. It provides a good tilth but suffers from the disadvantage that the tines often need replacing when it is used on stony soils.

The P & H Single-pass Stabilizer

15·24 This is a large machine (over 10 ft wide) which has been developed in the U.S.A. and has been used very successfully there (see Plate 15·6). Four contra-rotating rotors, housed under one hood which can be raised and lowered hydraulically, are driven by a 260 H.P. power unit. The machine advances slowly (at speeds up to 30 ft/min.) and the soil is cut out, pulverised and mixed in one pass of the machine. It has separate spray systems for adding water and fluid stabilizers to the requisite amount. Powdered stabilizers must be spread on the soil in front of the machine. Large outputs can be obtained, 8,000 sq. yd/day having been maintained on one job. The machine can deal with soils containing stones of maximum size 3 in.

TRAVELLING PLANT METHOD

15·25 Using travelling plant, the procedure is the same as in the mix-in-place method as far as the application of the stabilizer is concerned. In the case of cement, the pulverized soil is heaped into a windrow by a specially converted motor-grader or a sizer and the cement is spread on top. Occasionally the cement is spread before windrowing but this is unusual. In the case of fluid stabilizers, the soil is windrowed and the stabilizer added by the travelling mixer which, as in cement stabilization, moves along the line of the windrow. There are several types of mixer, two of which are described below:—

The Barber-Greene Mixer

15·26 (see Fig. 15·2 and Plate 15·7A) The soil and stabilizer are lifted by a bucket elevator and discharged into a hopper. The material then enters a pug-mill through an adjustable apron gate, and water (or a fluid stabilizer if required) is added and mixed into the soil. The mixture is then discharged

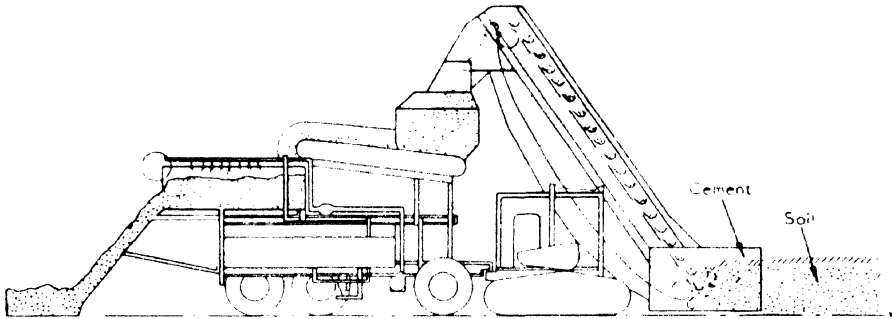
on to the road and spread with a grader. The subsequent procedure is the same as that for mix-in-place methods. The Barber-Greene machine has been used for soil-cement construction in this country.

Windrowed material is at right. The bucket loader pulls the mixer and picks up the soil to discharge it into the mixer hopper in centre. Calibrated gates on hopper release predetermined flow of soil. Jets shown over pugmill are for adding water. Soil is mixed in pugmill and discharged in rear of plant in windrow ready for spreading.

FIG. 15·2 SCHEMATIC DIAGRAM OF TRAVELLING MIXER FOR SOIL STABILIZATION

The Gardner Type Mixer

15·27 (see Plate 15·7B) This is a converted blade-grader, in which the windrow is scooped into a pug-mill (which replaces the blade of the grader) where water is added and mixing takes places. From the pug-mill the mixed material is deposited back in a windrow. It is desirable to have three mixers on he job, the first of which mixes the soil with the stabilizer, the second sprays water and mixes it in, and the third machine completes the mixing. The windrow of mixed material is spread with a motor grader and compacted and finished in the normal way. This method gave the enormous daily output of 37,000 sq. yd on an American airfield.

The Wood Road-mixer

15·28 (see Plate 15·8A) This is built on similar lines to the Gardner mixer, and has been used to a small extent in this country.

The Jaeger Machine

15·29 This machine works on the same principle as the Gardner mixer, but is mounted on tracks instead of wheels.

The H & B Motopaver

15·30 This operates on similar principles to the Barber-Greene machine, and has been used in the U.S.A., principally for bituminous stabilization.

STATIONARY PLANT METHOD

15·31 There are two main types of stationary mixing plant:—

(1) Continuous mixers.
(2) Batch mixers.

Continuous Mixers

15·32 The principle of the continuous mixer is similar to that of the travelling mixer. An elevating loader supplies material to a hopper with a measuring gate, whence a belt conveyor discharges it to a pug-mill, where water or a fluid stabilizer may be added through spray nozzles, and mixed into the soil. The mixed material is then discharged into lorries. Immobilized travelling plant may easily be arranged to serve as a central mixing plant. The size of a central mixing plant, built as such, will depend on the output required.

Batch Mixers

15·33 On small jobs, and especially for patching, concrete mixers have been used for mixing coarse-grained soils with stabilizers such as cement or bitumen emulsion. The procedure is identical with that employed in mixing concrete (Plate 15·8B), and even ordinary tilting-drum concrete mixers have been used with success. The best results, however, are given by double-paddle mixers, pug-mills or roller pan type machines, in which soil lumps are easily broken up.

15·34 The time required to obtain a uniform mixture depends on the type of mixer and the type of soil. The mixed material can with advantage be discharged vertically from the mixer into lorries, and it can then be transported directly to the site, tipped, spread and compacted in the normal manner. For compacting, finishing and patching small areas, hand or pneumatic tampers are useful.

COMPARISON OF DIFFERENT METHODS OF CONSTRUCTION

15·35 The advantages and disadvantages of the three methods are briefly discussed below.

Mix-in-place method

15·36 The advantages *are:—

(1) The plant is simple, cheap and easily transported.
(2) The number of machines required can be adjusted to the size of the job.
(3) The whole processed section is ready for compaction at the same time.
(4) A large average output may be maintained.
(5) In a wet climate the loss of water by evaporation may be advantageous, it is the only way of getting rid of excess moisture.

15·37 The disadvantages are:—

(1) It is not easy to obtain a uniform thickness of treatment, because of the difficulty of setting the machines to a given depth.
(2) The mixing is not as uniform as with travelling or stationary mixers.

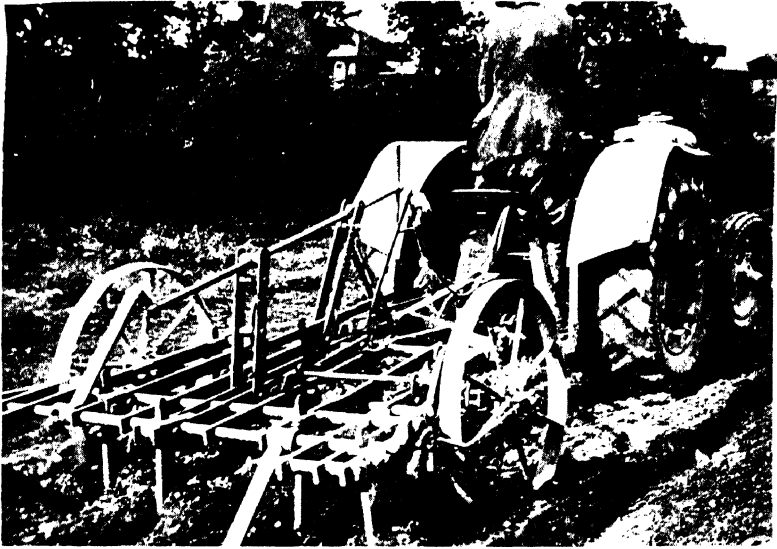
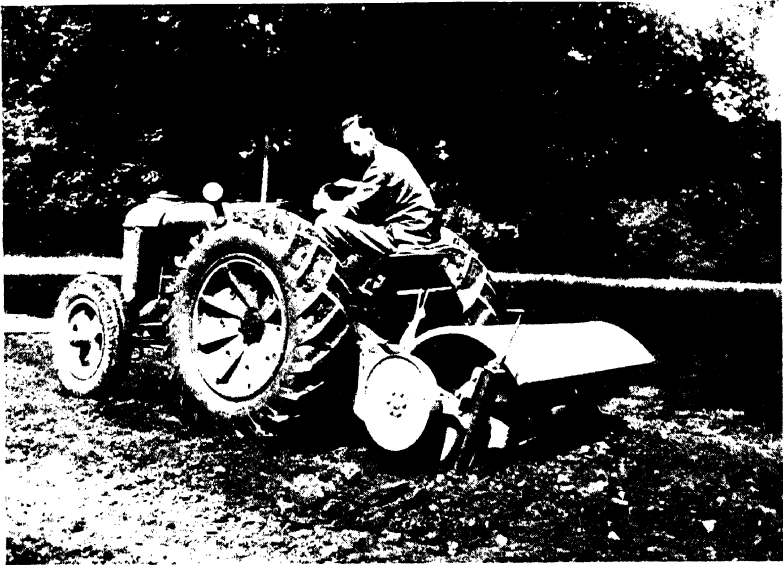
*These comments apply to the use of agricultural plant (including rotary tillers). Large single-pass machines, such as the P & H stabilizer, while very expensive and not easy to transport, are free from most of the disadvantages mentioned in the following list.

(A) SCARIFYING SOIL WITH A RIGID-TINE CULTIVATOR

(B) PULVERIZING SOIL WITH A ROTARY TILLER

PLATE 15·1

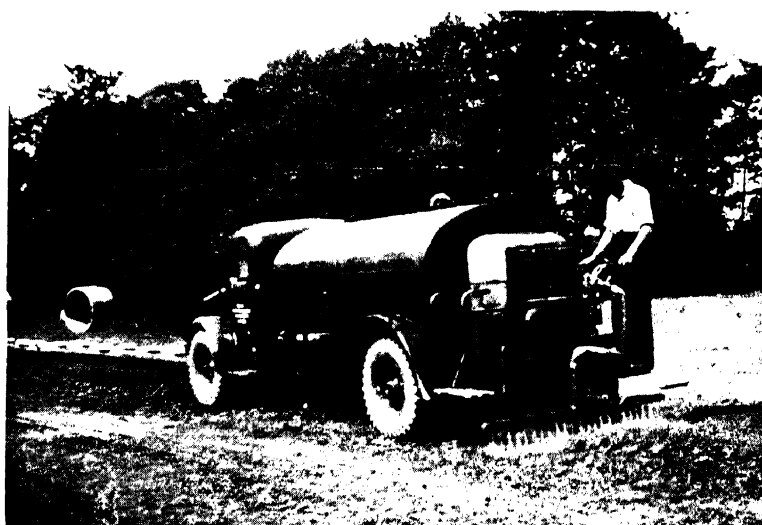
(A) ENSURING EVEN DEPTH OF TREATMENT WITH A GANG-PLOUGH

(B) APPLYING WATER FROM A SPRAY-TANKER

PLATE 15·2

(A) APPLYING CEMENT BY HAND

(B) APPLYING CEMENT FROM A BULK CEMENT SPREADER

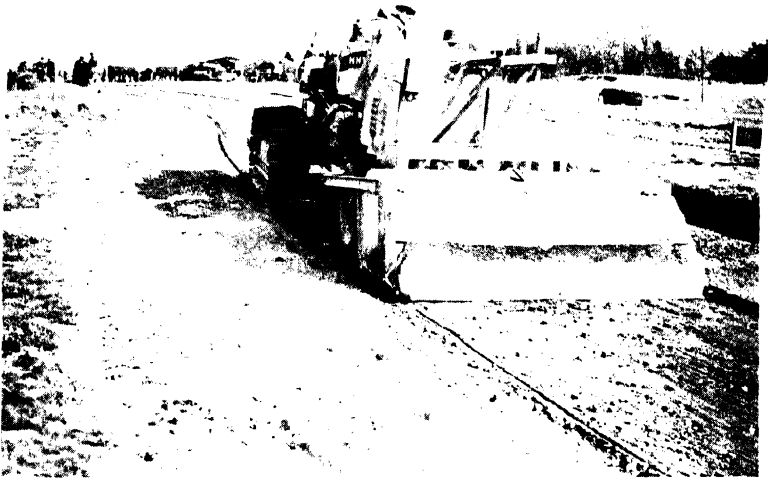
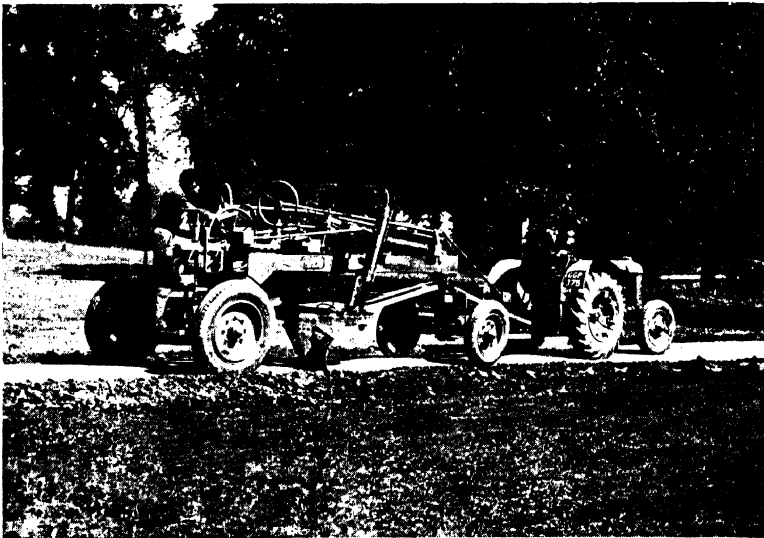



PLATE 15·3

(A) MIXING IN STABILIZER BY MEANS OF A PULVIMIXER

(B) SHAPING THE ROAD PROFILE WITH A BLADE-GRADER

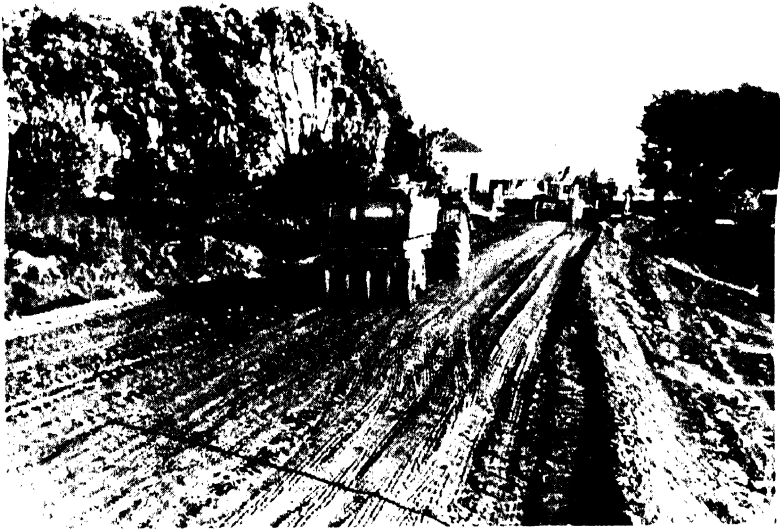



PLATE 15·4

(A) COMPACTING STABILIZED SOIL WITH A PNEUMATIC-TYRED ROLLER

(B) COMPLETED UNSURFACED STABILIZED-SOIL ROAD

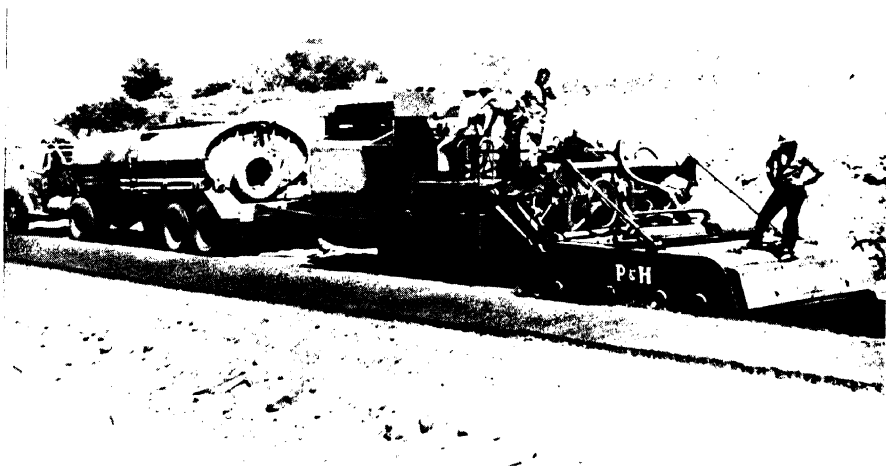
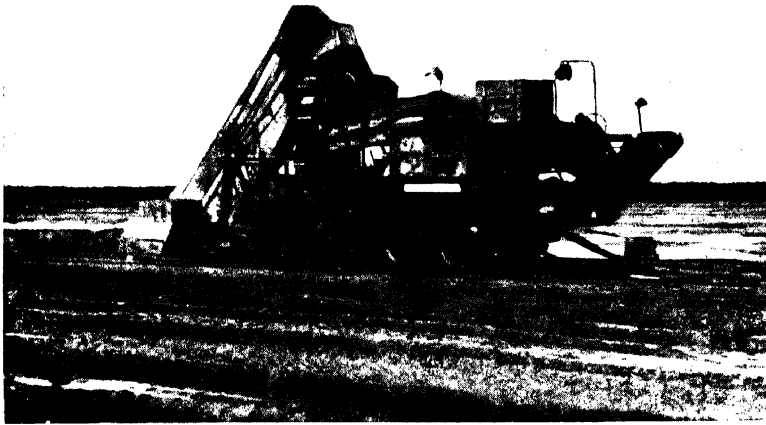
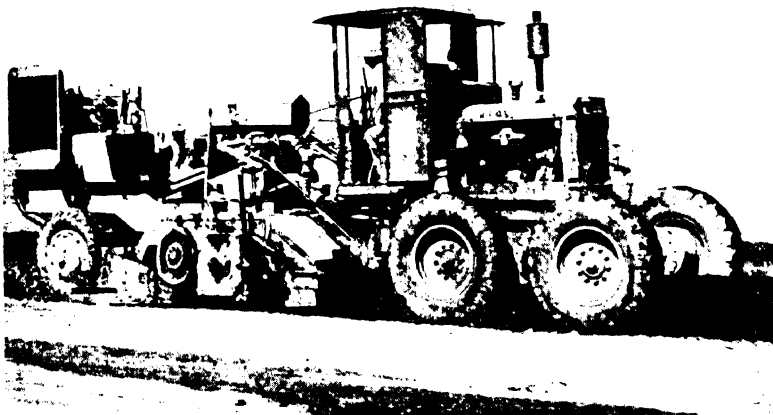



PLATE 15·5

P AND H SINGLE-PASS STABILIZER

PLATE 15·6

(A) BARBER-GREENE TRAVELLING MIXER

(With acknowledgements to PACIFIC BUILDER AND ENGINEER, the original source of the photograph)

(B) GARDNER TRAVELLING MIXER

PLATE 15·7

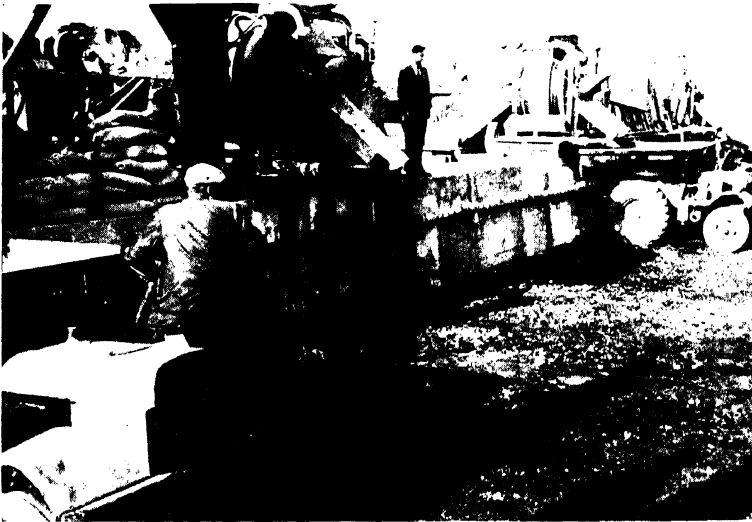
(A) WOOD ROAD-MIXER

(B) CENTRAL MIXING PLANT STANDING ON SOIL-CEMENT PLATFORM WITH SOIL-CEMENT RETAINING WALL
(Concrete mixing carried out with farthest mixers)

PLATE 15·8

(3) Heavy rain is liable to spoil a whole section. (0·1 in. rainfall corresponds to an increase of 1 to 1·2 per cent in moisture content for a layer 6 in. thick.)
(4) In a dry climate the water lost by evaporation is difficult to replace.

Travelling Plant Method

15·38 The advantages are:—
(1) Accurate proportioning of added water.
(2) Uniform mixing.
(3) Short mixing time.
(4) A uniform subgrade surface can be obtained, and the depth of treatment can be controlled.
(5) It has the highest output for a given expenditure of plant and labour.

15·39 The disadvantages are:—
(1) High initial cost of plant.
(2) The consequent need for the plant to work continuously at full capacity.
(3) Work may be stopped for a minor breakdown on one piece of plant.

Stationary Plant Method

15·40 The advantages are:—
(1) Accurate proportioning of the mixture.
(2) Easy control of depth of treatment.
(3) Concrete mixers can be used.
(4) No additional haulage if soil has to be taken from a borrow-pit.
(5) Small losses of moisture during mixing and transport of material.
(6) The method is suitable for use with form work, for instance when vibrators are required to compact uniform sands or when the stabilized layer is to form a sub-base to machine-laid concrete.

15·41 The disadvantages are:—
(1) Expensive if soil *in situ* is processed.
(2) Material must be compacted as delivered, and not as a complete section.

Typical average daily outputs given in the technical literature for the different methods are indicated in Table 15·1.

TABLE 15·1

OUTPUT PER 10-HOUR DAY FOR THE THREE CONSTRUCTION METHODS (6-IN. THICKNESS EXCEPT WHERE OTHERWISE INDICATED)

Method	Average daily output (sq. yd)	Maximum daily output (sq. yd)
Mix-in-place	2,000—8,000	13,000
Travelling plant	20,000—30,000*	60,000
Stationary plant	500—2,000	—

*4-in. thickness.

FIELD CONTROL

15·42 During construction it is necessary to check the following items:—

Degree of pulverization (mix-in-place).
Moisture content (all methods).
Dry density/moisture content relationship (all methods).
Cross-sectional area of windrow (travelling plant).
Depth of treatment or spreading (all methods).
Quality of mixed material (all methods).
Dry density of compacted layer (all methods).
Stabilizer content (all methods).

Degree of Pulverization

15·43 Before the stabilizer is added, the soil should be pulverized to such an extent that 80 per cent will pass a $\frac{3}{16}$-in. B.S. sieve, excluding coarse aggregate. To check this, representative samples are sieved during the pulverization process.

Moisture Content

15·44 The determination of moisture content is the control test most frequently required in stabilized construction. The procedure is to take samples of the pulverized soil every 300 to 400 ft along the centre-line of the site, and determine the moisture content of each. Tentative estimates may then be made of the additional water required, if any, so that watering can start as soon as the stabilizer is partly mixed with the soil. After the soil and stabilizer have been mixed, samples are again taken at the same points as before. This allows an exact estimation to be made of the water that is finally required. Check tests are made at the same points towards the end of mixing of any water added.

15·45 During compaction, the moisture content is again determined as a check that the required value is being maintained, and in particular to detect any drying of the surface. Table 15·2 summarizes the stages at which moisture

TABLE 15·2

STAGES AT WHICH MOISTURE CONTENT DETERMINATIONS ARE DESIRABLE DURING MIX-IN-PLACE CONSTRUCTION

No.	Stage of the work	Object
1	Pulverization of soil	To check whether soil is within 2% of the specified moisture content
2	Beginning of day's work	To obtain approximate estimate of water requirements
3	At the completion of mixing of soil and stabilizer	(a) To determine the exact amount of water required (b) To estimate the rate of evaporation
4	At the completion of mixing any extra water added after 3	Check tests
5	During compaction	Check tests to ensure surface has not dried

contents should be determined for mix-in-place work in the British Isles. A different plan may be desirable under different climatic or other circumstances.

15·46 With travelling plant the sequence is rather similar, but with stationary plant it is probably sufficient to control the moisture content of the incoming soil and the mixed material produced at hourly intervals from a given machine. The techniques for determining moisture content in the field are described in detail in Chapters 3 and 13.

Dry Density/Moisture Content relationship

15·47 This varies with the soil type and it may be desirable to carry out tests from time to time during the work, to make allowance for any variation from the results of the preliminary tests in the laboratory.

Cross-sectional Area of Windrow

15·48 For travelling plant working on a windrow, it is necessary to measure the cross-sectional area of the windrow to ensure that the proportioning of stabilizer is correct, and that the correct thickness will be laid.

Depth of Treatment or Spreading

15·49 If mix-in-place construction is used, the depth of treatment must be controlled from the beginning of the processing, and the pulverizing and mixing equipment must be carefully adjusted. If stationary plant is used, the thickness of treated material can be controlled by spreading it with a bulldozer, the blade being supported on runners. The depth of a compacted layer can be checked in conjunction with measurement of dry density (see Chapter 9). American specifications require test holes to be dug at intervals not exceeding 500 ft, and that the compacted thickness at any place should be within ¾ in. of that specified, and these appear to be suitable for British practice.

Quality of the Mixed Material

15·50 It is very useful to be able to check the quality of the stabilized material as mixed by the field plant and to compare this with the quality of the stabilized material produced in the laboratory, upon which the design of the work was based. This can be done by making up specimens from the mixes in the field in a similar manner to the manufacture of laboratory specimens and by applying the normal tests appropriate to the stabilizer used. Thus cement-stabilized specimens would be tested for compressive strength after 7 days, and specimens stabilized with bituminous material would undergo a capillary water absorption test. These tests are described in Chapters 12 and 13. Samples should be taken at intervals of 100-500 ft along the road. It is useful to take a series across the road, in addition, to ensure that the mixing has not been concentrated at the centre of the lane.

Density of the Compacted Layer

15·51 The dry density of the stabilized soil is generally measured by the sand-replacement method every 100 to 500 ft. Dry density checks should be carried out after each day's construction to enable adjustments in the compacting procedure to be made. No dry density should be below that specified by more than 5 lb./cu.ft.

Stabilizer Content

15·52 In all three methods of construction it is desirable to check the stabilizer content of the mixed material, if this is possible. Running control of this factor enables an excess or a deficiency of the stabilizer to be corrected as soon as possible.. Unfortunately, the present methods of determining stabilizer contents are somewhat complex and require the presence of a site laboratory and a skilled analyst, and are therefore only justified on medium to large jobs. On smaller jobs, it is often useful to take representative samples for testing at a central materials laboratory. The methods for determining the percentages of cement and bituminous stabilizers are described in Chapters 12 and 13.

SUMMARY

15·53 Stabilized soil can be prepared in the field in three ways, viz. (1) by mix-in-place methods, (2) by travelling plant, or (3) by means of stationary plant. Descriptions of the three methods are given, the advantages and disadvantages compared, and typical outputs indicated. Careful control is needed in stabilized construction and amongst the items that need checking are the degree of pulverization of the soil, its moisture content, the depth of construction and the stabilizer content.

CHAPTER 16

SOIL MOISTURE AND THE FACTORS GOVERNING ITS MOVEMENT

INTRODUCTION

16·1 If the subgrade of a road becomes wetter or drier after construction it may swell or shrink and change in strength: this may cause the surface of the road to deteriorate. A road built on a clay subgrade may, for example, fail owing to inadequate strength, following a comparatively small increase in the moisture content of the clay. On the other hand, the drying of the soil near the verges during a period of abnormal drought may result in settlement of the subgrade and longitudinal cracking of the road surface. Water moving through the soil under the action of gravity can be intercepted or removed from the subgrade by suitably placed subsoil drains. Some of the water is retained in the soil by surface forces and cannot be removed by drains; the distribution of this water is determined by the equilibrium conditions of suction and vapour pressure in the soil. The quantity of water held in this manner increases with the clay content of the soil and may exceed 50 per cent of the dry weight in heavy clays.

16·2 Before attention can be given to the question of preventing changes in the subgrade moisture distribution all the factors which govern the movement of both free and held water in soil must be considered. In this chapter these factors are reviewed in detail and a mathematical treatment of those aspects of the thermodynamics of soil moisture which are of interest to the soils engineer, is given in the appendix to this chapter.

CLASSIFICATION OF SOIL WATER

16·3 To the engineer soil consists of a mass of mineral particles, variable in size, shape, arrangement and degree of compaction, between which there exists an intricate network of pores each connected with others by channels of different sizes. These channels communicate eventually either with the surface of the soil, or with cracks and fissures in the soil structure.

16·4 Of the water which falls on the soil under natural conditions, some passes through to form a water-table on some impermeable stratum below. The water which passes through the soil in this manner is generally referred to as " gravitational water," and the water below the water-table is termed " ground water." When the supply of surface water and the flow of gravitational water cease, some moisture is retained in the smaller pores and channels, and on the surface of the particles by surface tension and adsorptive forces. This water, which cannot be drained directly, may be conveniently termed " held water." The water vapour which fills the soil interstices not occupied by water in the liquid phase, and which under certain circumstances may

play an important part in determining the distribution of moisture in the soil, can be regarded as constituting part of the held water. This broad classification of soil water, which is summarized in Fig. 16·1, is considered in greater detail in the following paragraphs.

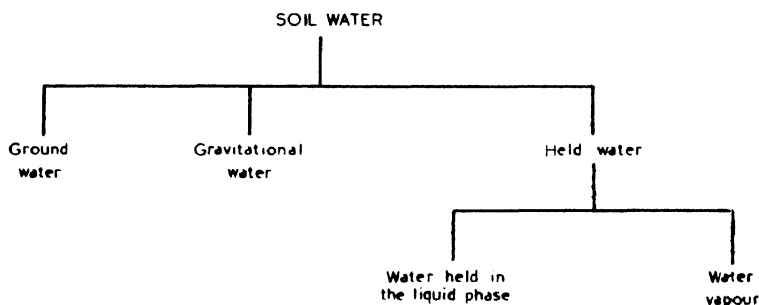



FIG. 16·1 BROAD CLASSIFICATION OF SOIL WATER

GROUND WATER

16·5 Beneath the water-table the soil pores are completely filled with water and any movements of ground water which occur as a result, for example, of steps taken to lower the water-table, are assumed to follow Darcy's law of saturated flow. This law states that the velocity of flow through a column of saturated soil is proportional to the hydraulic gradient. It follows from Darcy's law that the quantity flowing through such a column in unit time is proportional to the area of the column, and the hydraulic gradient,

i.e. $Q = K.i.A.$ where i is the hydraulic gradient, and Q the quantity of water flowing in unit time through a column of total cross-sectional area A.

16·6 The constant of proportionality K, termed the coefficient of saturated permeability, can therefore be defined as the velocity of flow for unit hydraulic gradient, the velocity of flow being based on the total area of the soil through which flow occurs, and not on the average area of the voids. Fig. 16·2 shows a simple method for measuring the saturated permeability of soil. In this arrangement the hydraulic gradient is h/l, and if Q is the quantity of water flowing through the soil in unit time,

$$K = \frac{Q}{A} \cdot \frac{l}{h}$$

During tests the head must be maintained by a suitable inflow and for this reason the arrangement is generally referred to as a constant-head permeameter. The apparatus lacks sensitivity, and can only be used for coarse-grained soils. For fine-grained soils, variable-head permeameters, in which the flow of water is metered at the inflow end, are more reliable. The principle is shown in Fig. 16·3. If h_1 and h_2 are the heights above the free water level of the water in the narrow-bore inflow tube at times t_1 and t_2, K is given by the equation :—

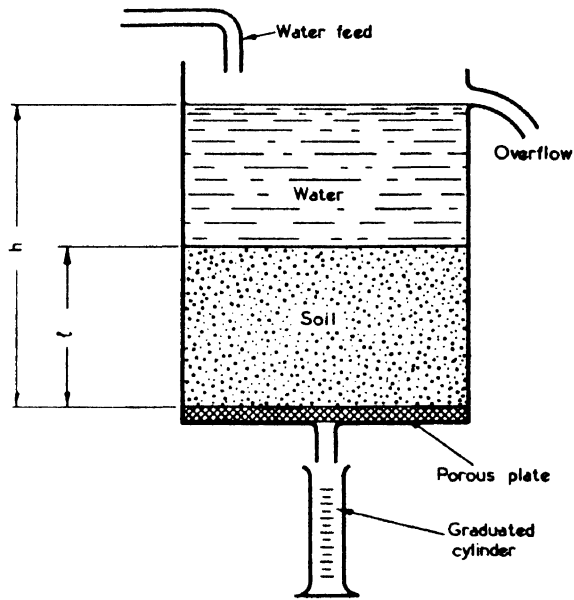



FIG. 16·2 CONSTANT-HEAD PERMEAMETER

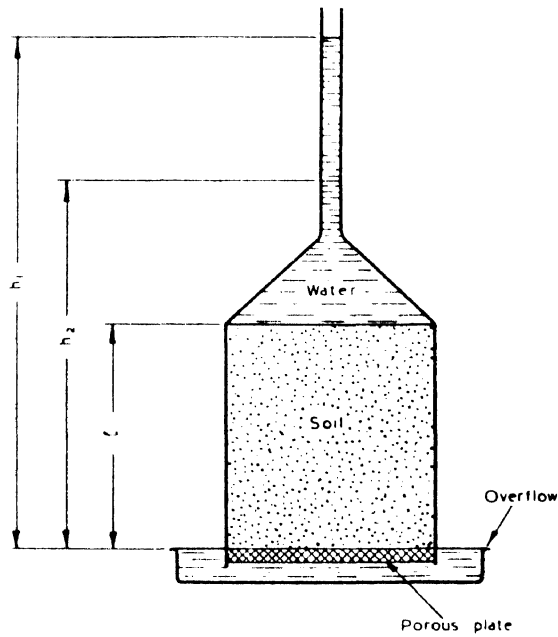



FIG. 16·3 VARIABLE-HEAD PERMEAMETER

$$K = \frac{2 \cdot 303\, al}{A\,(t_2 - t_1)} \log \frac{h_1}{h_2}$$

where a and A are the respective cross-sections of the narrow tube and the soil sample.

16·7 The value of K depends primarily on the particle-size distribution of the soil; in gravels it may exceed 20,000 ft per day and in heavy clays be less than 0·0002 ft per day. Its value also depends on the structure of the soil. It is necessary, therefore, to carry out permeability tests on undisturbed samples. Further, since the permeability in the vertical and horizontal directions may differ, samples should be cut in the direction in which flow occurs. To the road engineer, the value of K is useful for estimating the flow of water to drains installed to lower a high water-table.

GRAVITATIONAL WATER

16·8 The movement of gravitational water also depends largely on the structural characteristics and the porosity of the soil but, owing to the presence of air in the soil pores, Darcy's law can no longer be applied. Gravitational water is not directly of interest to the road engineer, unless during its passage to the water-table it is deflected into the subgrade by interposed layers of impermeable material, e.g. side-long seepage. Where there is a possibility of gravitational water being deflected in this manner intercepting drains are necessary.

HELD WATER

16·9 Although the water held in soil does not move freely under the action of gravity, it cannot be regarded as static. In general the movements which occur are slow, but considerable quantities may be transferred over long periods, both in the liquid and vapour phases. The mechanism by which held water is retained in the soil and the factors which govern its movement are considered below.

Classification of Held Water (excluding water held in the vapour phase)

16·10 Water held in soil, excluding water vapour, can be divided into four categories. Arranged in order of the tenacity with which they are held, these categories are: (1) water chemically combined in the crystal structure of the soil minerals, (2) water adsorbed on the surface of the particles, (3) water held by surface tension round the points of contact of the particles, and (4) water held by capillarity in the pores between the particles.

Chemically Combined Water

16·11 The water combined in the crystal structure of the soil minerals is very small in quantity and cannot be removed by drying the soil at 110°C. It can be regarded from the engineering viewpoint as an integral part of the soil solids.

Adsorbed Water

16·12 The water held by forces of adsorption on the surface of the soil particles can be reduced by oven-drying, but not entirely removed. An oven-dry soil, if exposed whilst cooling, will adsorb water, the rate of adsorption depending on the humidity of the surrounding air. The maximum quantity of water held

in a soil by surface adsorption depends primarily on the surface area of the particles. Further consideration is given to adsorbed water during the discussion of movements of moisture in the vapour phase.

Water held by Surface Tension and Capillarity

16·13 The greater part of the held water in granular soil is retained by surface tension either round the points of contact of the particles or in the soil pores and capillaries. The manner in which this water is held is perhaps best approached by a brief discussion of surface tension.

16·14 If a single drop of water falls on a sheet of clean glass (or other substance " wetted " by water) surface tension forces act at the boundary of the two materials tending to spread the water over the surface of the glass. This force is present at any liquid/solid boundary, irrespective of the curvature of the surface of the solid. A drop of water introduced at the point of contact of two spheres will, for example, be held in equilibrium by surface tension in the manner shown in Fig. 16·4. In this case the air/water interface has two principal curvatures, the radii of which both increase if additional water is added. (These curvatures suffer slight deformation due to the action of gravity according to the way in which the spheres are oriented.) For such an arrangement, there is a limiting quantity of water which can be held against gravity. Any water additional to this quantity drains away over the surface of the spheres.

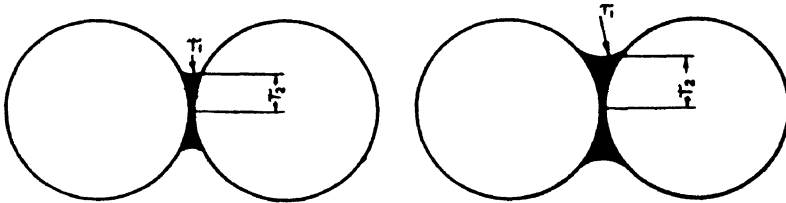



FIG. 16·4 WATER HELD BY SURFACE TENSION ROUND THE POINT OF CONTACT OF TWO SPHERES
(shown in section)

16·15 If the lower end of a glass tube of narrow bore is immersed in water (Fig. 16·5(a)) the level of the water in the tube rises above that of the surrounding liquid, again owing to surface tension acting at the boundary of the water and glass. Since the liquid in the tube is in equilibrium, it follows that the upward component of the surface tension force is equal to the gravitational force acting on the suspended liquid. Hence,

$$2T \cos \alpha = \gamma_w hgr$$

$$\text{or } h = \frac{2T \cos \alpha}{\gamma_w rg}$$

where T is the surface tension per unit length of the boundary, r the radius of the tube, and γ_w the density of water and h the capillary rise.

Since for glass and water the contact angle $\alpha = 0$,

$$h = \frac{2T}{\gamma_w rg}$$

From this equation it follows (1) that the height of rise increases as the diameter of the tubes decreases (Fig. 16·5 (b)), and (2) the radius of the tube, and hence the radius of the water meniscus, is inversely proportional to the reduction of pressure across the meniscus; the pressure reduction or suction in the water immediately below the meniscus being $\gamma_w hg$ (below atmospheric pressure). The actual variation of hydrostatic pressure, with respect to the pressure at the free water surface, is shown in Fig. 16·5(c).

16·16 Equilibrium is not possible in the case of a vertical water column not in contact with a free water surface (Fig. 16·5(d)), since the surface tension at the two ends of the column are equal and there is no resultant force to oppose gravity. Equilibrium is, however, possible in a tube of non-uniform section (Fig. 16·5(e)) or in the case of a " necked " tube, where the downward component of the surface tension may be smaller than the upward component.

16·17 Soil minerals are " wetted " by water in a similar manner to glass, and although the soil particles cannot be regarded as spherical or the channels connecting the pores as circular capillaries, the results deduced above facilitate a qualitative understanding of the manner in which water is retained in the soil by forces due to surface tension.

16·18 The sequence of events which occurs when the moisture content of an oven-dry soil is slowly increased can be envisaged as follows. The first water added to the soil thickens the adsorbed water films. Water then begins to collect round the points of contact of the particles, the radii of the air/water interfaces slowly increasing as more water is added. The openings between the soil pores, which can be regarded as " necked " capillaries, begin to fill as the water rings round the particles coalesce. For each such opening there will be a maximum quantity of water which can be held against gravity. When this quantity is reached, water drains away across the surface of the particles to combine with water in lower openings not yet filled to capacity. Eventually a condition is reached when no further water can be held, and additional water supplied passes through the soil. This is the gravitational water already discussed.

16·19 Where a water-table is present in the soil, the channels connecting the individual pores can be regarded as a mass of irregular capillary tubes, not necessarily vertical, and of differing diameters, connecting with the water-table. It follows that water will rise to different heights in the various channels, resulting, in incompressible soils, in a general decrease in moisture content as the height above the water-table is increased. Even the most approximate calculations of the moisture contents above the water-table are, however, precluded by the irregular nature and distribution of the channels. The problem of the distribution of the moisture above a water-table, one of particular importance to the road engineer, is best approached by considering the soil suction/moisture content characteristics of soils.

Suction of Held Water

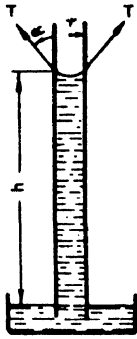
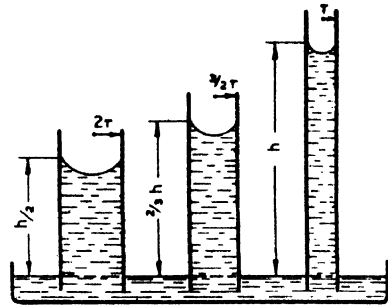
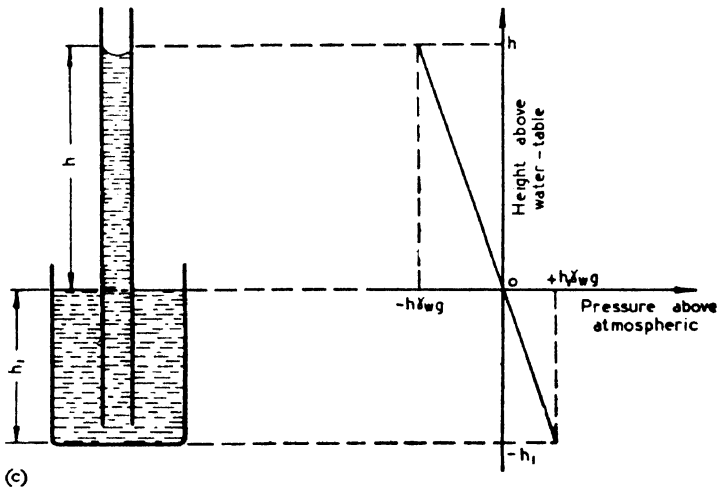
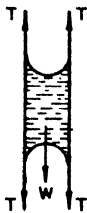
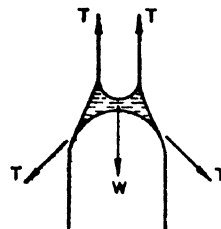
16·20 The held water in soil is retained in a state of reduced pressure or suction which may be termed the soil moisture suction or more briefly the soil suction. In the capillary tube analogy it was seen that the water held above the water-

(a) Rise of liquid in capillary tube

(b) Effect of radius of tube on height of capillary rise

(c) Variation of pressure in liquid above and below water-table

(d) Equilibrium of liquid not possible—liquid drains

(e) Equilibrium of liquid possible—liquid held

FIG. 16·5 THE EQUILIBRIUM OF A LIQUID IN NARROW-BORE TUBES

table had a pressure lower than that at the free water surface, the pressure difference at the top of the column being related to the radius of the meniscus. It has also been seen that the air/water interfaces throughout the soil consist of menisci, the curvature of which indicates in a similar manner the state of reduced pressure or suction in the soil water. As the moisture content of the soil is reduced, and the water interfaces recede into the smaller pores, their radii of curvature decrease, indicating an increase in the soil suction.

16·21 It has been found experimentally that the increase in soil suction with decreasing moisture content is continuous over the entire moisture range. Its value rises from zero at saturation to many thousands of pounds per square inch in oven-dry soil. This large variation makes the use of a logarithmic scale essential if the soil suction/moisture content relationship is being considered as a whole. In this connexion the pF scale, introduced by Schofield[1], is frequently used. If the soil suction is expressed in terms of the length of a suspended water column, the common logarithm of this length expressed in centimetres of water is equivalent to the pF value of the soil moisture. Table 16·1 shows the relationship between the pF scale and soil suction expressed in lb./sq.in. It will be observed that, owing to the logarithmic nature of the unit, pF = 0 does not correspond exactly to zero suction.

TABLE 16·1

RELATIONSHIP BETWEEN pF AND SOIL MOISTURE SUCTION EXPRESSED IN LB./SQ. IN.

pF	Equivalent suction	
	cm. of water	lb./sq. in.
0	1	0.0142
1	10	0.142
2	100	1.42
3	1000	14.2
4	10000	142
5	100000	1420
6	1000000	14200
— ∞	0	0

16·22 Fig. 16·6 shows the relationship between soil suction and moisture content for a loam soil[1]. It will be seen that the suction when the soil is being " wetted," i.e. when the moisture content is increasing, is lower than the suction at the same moisture content when the soil is drying. This hysteresis effect may arise because the release of water from the larger pores is to some extent controlled by the surrounding smaller pores, during the drying process.

The Measurement of Soil Suction

16·23 Four methods of measuring soil suction are readily available within the range of interest to the soil engineer. These are (1) the tensiometer, (2) the direct suction, (3) the suction plate and (4) the centrifuge methods.

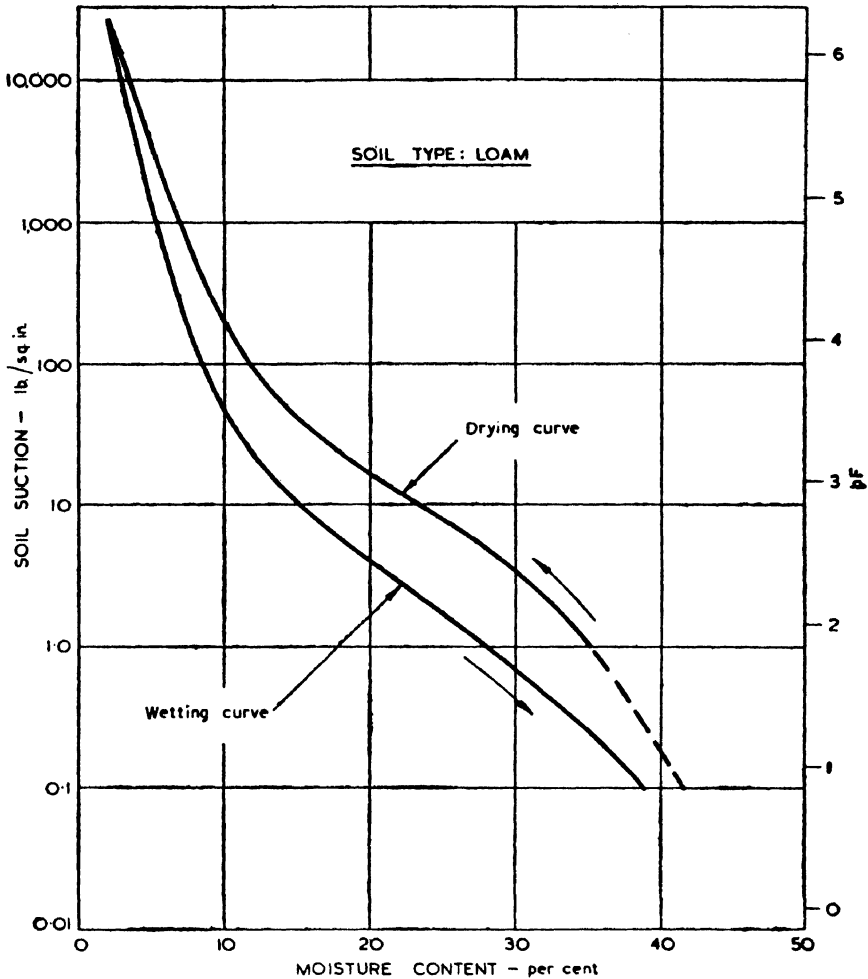



FIG. 16·6 RELATIONSHIP BETWEEN SOIL SUCTION AND MOISTURE CONTENT

Curves for wetting and drying conditions of the soil (Schofield)

16·24 In the tensiometer method (Fig. 16·7) a porous pot filled with water, and connected by a water-filled tube to a mercury manometer, is placed in the soil. Water is drawn through the pores of the pot by the suction of the surrounding soil moisture until the suctions inside and outside the pot are equal. The equilibrium suction inside the pot is readily deduced from the manometer reading, whilst the moisture content of the soil round the pot is measured in the usual manner. The soil suction/moisture content relationship is explored using a range of initial soil moisture contents. This is a wetting method suitable for suctions up to about 12 lb./sq.in.

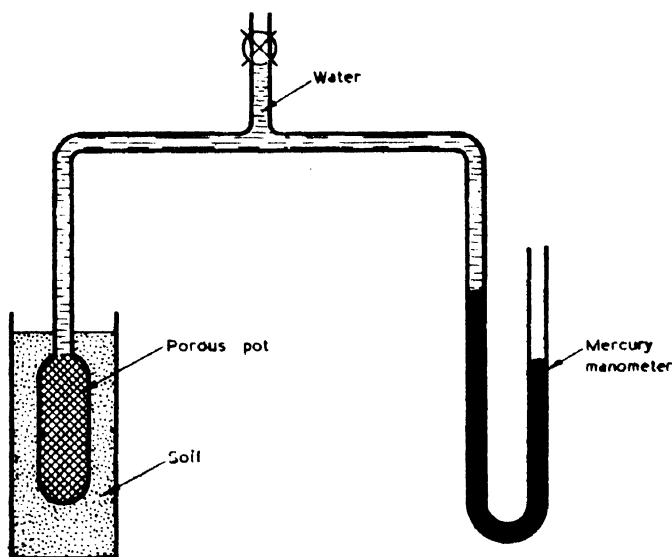



FIG. 16·7 PRINCIPLE OF THE TENSIOMETER METHOD OF MEASURING SOIL SUCTION

16·25 In the direct suction method (Fig. 16·8) covering approximately the same range, a thin sample of the saturated soil is placed in a Büchner type funnel, and subjected to a suction of known magnitude. Water leaves the soil until the soil suction rises to a value equal to the applied suction. The moisture content of the soil is determined when no further water is drawn away. By using a range of applied suction the soil suction/moisture content relationship for the soil in the drying condition can be determined. The specimen should be covered during the test to prevent evaporation.

16·26 The suction plate method (Fig. 16·9) is similar to the direct suction method except that the suction is provided in the plate hydrostatically. The porosity of the plate is such that the air will not pass through when it is being used for suctions less than about 12 lb./sq.in. The soil sample is placed on the plate and a transfer of moisture occurs until the plate and the sample are in suction and vapour equilibrium. The suction in the plate, and hence the vapour pressure in the enclosure, can be controlled at any value using the vacuum pump and its associated manometer. This method has the advantage over the direct suction method that the soil is not subjected during the test to an overburden due to the atmospheric pressure acting only on the upper face of the specimen. The method is applicable to wetting and drying tests.

16·27 Suctions of several hundred pounds per square inch can be obtained using a centrifuge method. A thin sample of the soil under test is placed in a cup of the type shown in Fig. 16·10. During the centrifuging process water leaves the soil and travels through the walls of the porous pot to the water-table, the level of which is maintained constant by the provision of a small escape hole. When equilibrium is reached, the suction of the moisture left

in the soil is sufficient to prevent any further migration to the water-table. The soil suction can then be calculated from the speed of the centrifuge using the formula:—

$$h = \frac{\omega^2}{2g}(r_1^2 - r_2^2)$$

where ω is the angular velocity, h is the suction expressed in terms of the height of the equivalent water column, i.e. log h = pF of soil moisture, and r_1 and r_2 are the respective distances of the water-table and the centre of the soil sample from the centre of rotation; h, r_1 and r_2 being measured in centimetres.

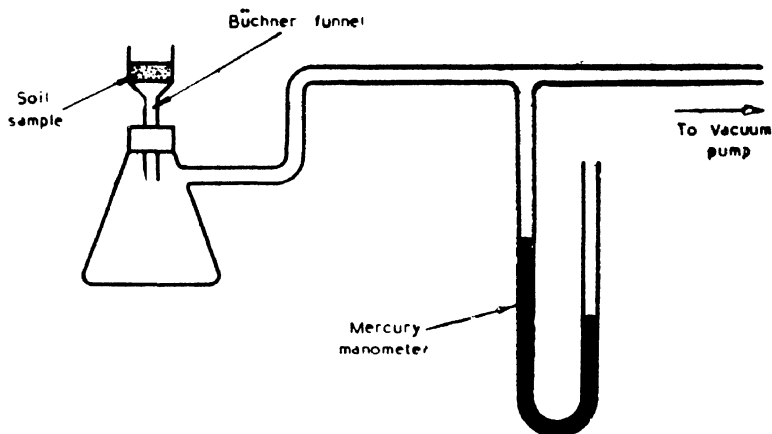



FIG. 16·8 PRINCIPLE OF THE DIRECT SUCTION METHOD FOR MEASURING SOIL SUCTION

16·28 Points on the relationship between soil suction and moisture content can be obtained by carrying out tests at various centrifuge speeds. During the centrifuging process the soil is subjected to an overburden pressure due to its weight. At high speeds this pressure may be considerable. It is for this reason that very thin samples should be used.

Factors governing the Soil Suction/Moisture Content Relationship

16·29 Fig. 16·11 shows soil suction/moisture content curves obtained at the Road Research Laboratory using the drying method on a range of remoulded soils having different clay contents. The moisture held at a fixed suction clearly increases with clay content. This is because the number of small channels in which water is retained, and also the total surface area of the particles, both increase as the proportion of finer constituents increases. The structure and bulk density of the soil also appear to affect the soil suction/moisture content relationship, but these factors have not yet been fully investigated. Deductions regarding the distribution of soil moisture under field conditions should, therefore, be based on tests carried out on undisturbed samples. Since the surface tension at an air/water interface decreases slightly with increasing temperature,

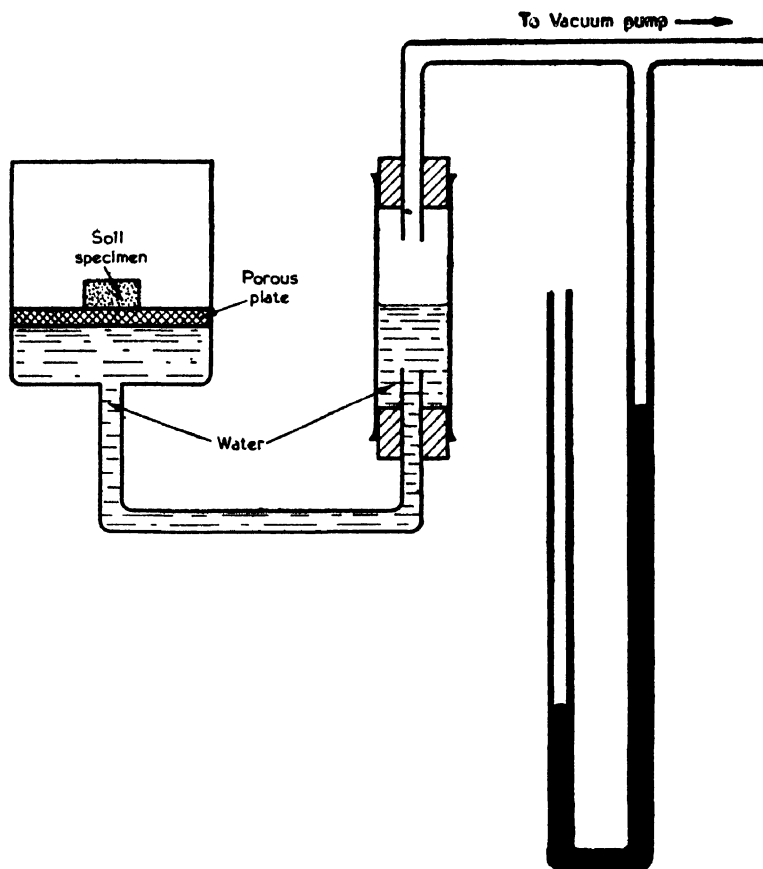



FIG. 16·9 PRINCIPLE OF THE SUCTION PLATE METHOD FOR MEASURING SOIL SUCTION

the suction of soil water decreases in a similar manner. The variation, which in general is small enough to be neglected, is considered in greater detail in the appendix to this chapter.

Movement of Moisture in the Liquid Phase

16·30 Held moisture moves in the liquid phase from regions of low suction to regions of higher suction. Vertical movements, however, are affected by gravity in the manner discussed below in connexion with the distribution of moisture above the water-table. An upward migration of moisture cannot occur in a soil between two regions having a vertical separation of h feet unless the soil suction at the higher region exceeds that at the lower by h feet of water. Neglecting this effect of gravity, in a soil of uniform type any local change in moisture content, by destroying the suction equilibrium, sets up movements of moisture tending to restore the moisture content at a uniform higher or lower value depending on the nature of the local change. It follows, however, from the curves shown in Fig. 16·11 that soils of different types may be in suction

equilibrium at widely different moisture contents. Samples of Culham sand and London clay (remoulded), would both have a suction of 1 lb./sq.in. and be in moisture equilibrium when their moisture contents were 19 per cent and 56 per cent respectively. It is for this reason that strata of sand found in clay during boring tests have a much lower moisture content than the surrounding soil.

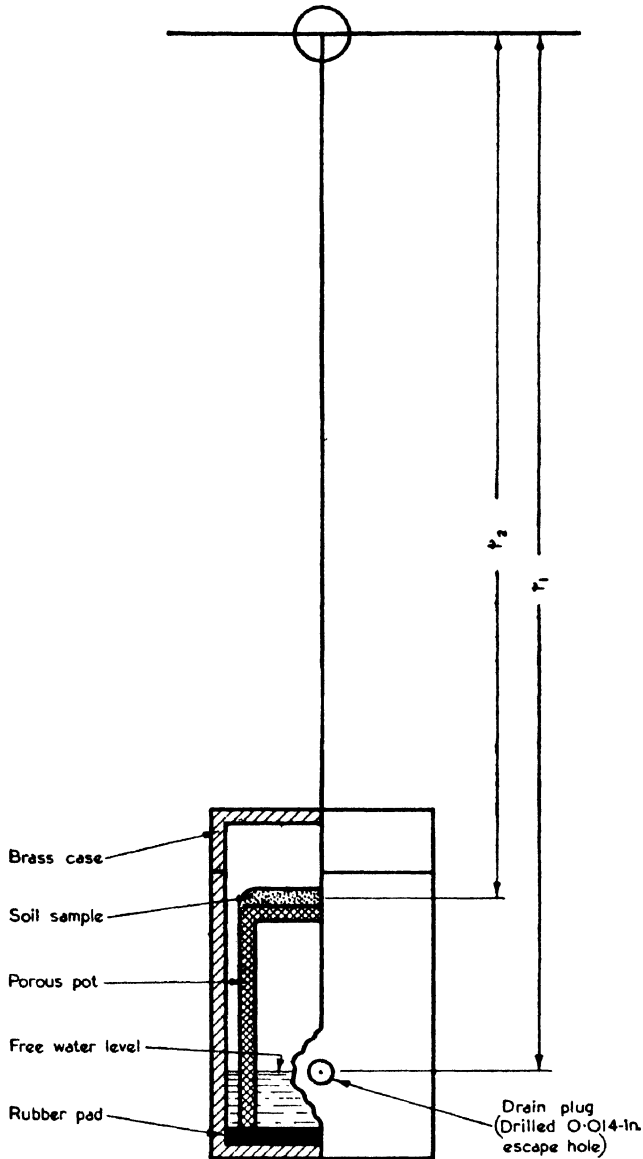



FIG. 16·10 PRINCIPLE OF THE CENTRIFUGE METHOD FOR MEASURING SOIL SUCTION

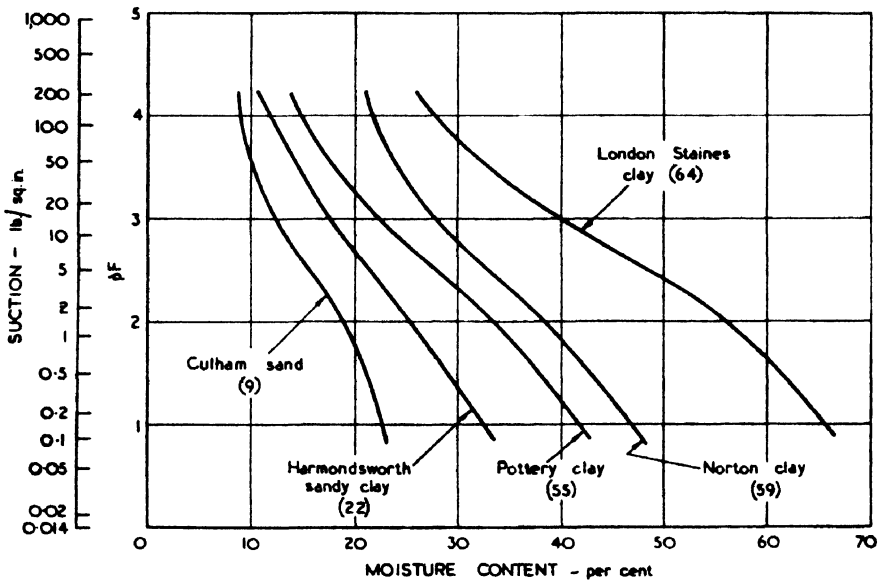



FIG. 16.11 COMPARISON OF SOIL MOISTURE SUCTION (DRYING) CURVES FOR FIVE SOILS

(Clay content of each soil shown under soil title)

16·31 It should be noted that if the upper strata of a subgrade have a moisture content materially below that of the soil beneath, when the road is surfaced movements of moisture will occur as a result of suction forces tending to increase the moisture content of the soil under the pavement. This is an important factor likely to cause regression in compacted subgrades.

16·32 When roads have exposed verges, the moisture content of the soil forming the verges is normally higher in winter than that of the subgrade. There is, therefore, a tendency for moisture to move into the upper layers of the subgrade during the winter months, followed by a reverse movement during the summer. Seasonal movements of slabs resulting from such moisture changes are common on concrete roads built on clay subgrades, whilst in periods of abnormal drought, such as during the summer and autumn of 1947, the moisture distribution in the subgrade may be sufficiently disturbed to cause severe cracking of flexible pavements.

16·33 The seasonal variations of moisture content in the surface soil, which normally extend to a depth of about 3 ft in this country should be taken into account when strength tests are being performed in connexion with road design. If such tests are carried out in summer, the exposed subgrade may be materially drier than the mean condition which it will eventually assume under the pavement, and an inadequate design may result. On the other hand, if the tests are carried out in winter (or if, as is sometimes done, soaked specimens are used) the design may be unnecessarily conservative. Unless circumstances are such that there is a serious risk of wet subgrade conditions developing, e.g. where

the water-table level cannot be satisfactorily controlled, it is suggested that the soil tests should be carried out on the soil at the moisture content found at a depth of about 3 ft.

Soil Moisture Suction and the Moisture Distribution above a Water-table

16·34 If in a granular soil the effect of overburden pressure is neglected the moisture distribution above the water-table can be obtained from the soil suction/moisture content relationship. At a height h above the water-table the water will be held at a suction equivalent to a column of water of height h and a pF value of $\log_{10}$ h if h is expressed in cm. The moisture content at any height can therefore be read off the suction curve for the soil. Further, the effect on the distribution, of changes in the level of the water-table can be readily deduced. Fig. 16·12 shows the soil suction/moisture content relationship obtained using undisturbed samples of a sandy clay soil. In Fig. 16·13 the curve showing the equilibrium distribution of moisture content based on Fig. 16·12 is shown, together with the actual values of moisture content obtained at various depths by boring and sampling. The experimental points agree closely with the curve except in the zone where the moisture content is affected by surface evaporation and vegetation. The approximate configuration of the air/water interfaces at various heights above the water-table, for a case such as that considered above, is shown diagrammatically in Fig. 16·14.

16·35 Consideration of the moisture distribution in heavy clay soils, which under field conditions are normally saturated, presents more difficult problems. If a sample of such a soil is subjected to a uniform applied pressure (a condition approximately satisfied in the consolidation test) water is expelled until the suction in the soil water in the specimen is equal to the applied pressure. It follows that for such a soil the suction/moisture content relationship could be obtained from the results of the consolidation test.

16·36 In practice any soil stratum below the surface is subjected to an overburden pressure due to the self-loading of the soil above and to any additional load on the surface. The suction/moisture content relationship has been used to determine the equilibrium distribution of moisture for any position of the water-table, the latter in this case being defined as the zone in which the pore water is at atmospheric pressure. For the case where the water-table is in the surface, calculations based on this approach indicate that the moisture content has a high value at the surface, falls rapidly in the first few feet, and thereafter decreases much more slowly to great depths. A much smaller change in moisture content occurs in the upper strata if the water-table is below the surface.

Water Vapour

16·37 This discussion of soil moisture has so far been confined to water in the liquid phase. Consideration must now be given to the water vapour occupying the soil voids, and to the part which vapour movement plays in determining the final distribution of held water.

Soil Vapour Pressure

16·38 Water placed in a small evacuated enclosure evaporates until the pressure in the water vapour reaches a certain value depending on the temperature, after which the rates of evaporation and recondensation at the water

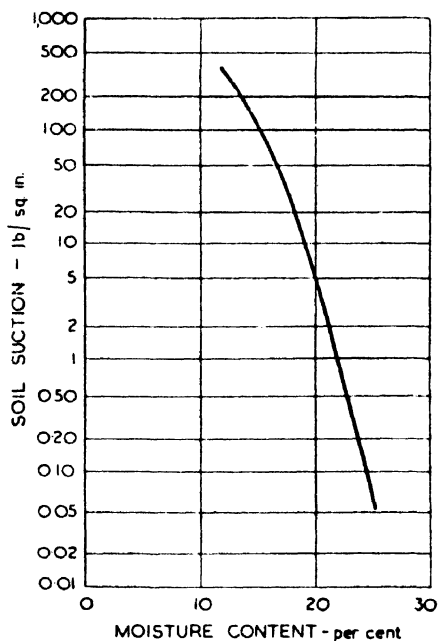



FIG. 16·12 SOIL SUCTION/MOISTURE CONTENT RELATIONSHIP FOR UNDISTURBED HARMONDSWORTH SANDY CLAY

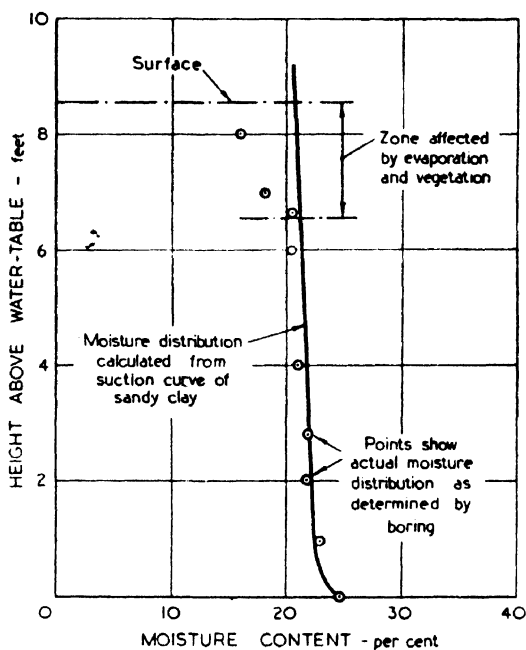



FIG. 16·13 COMPARISON BETWEEN MOISTURE DISTRIBUTION CALCULATED FROM SUCTION CURVE OF FIG. 16·12 AND ACTUAL MOISTURE DISTRIBUTION

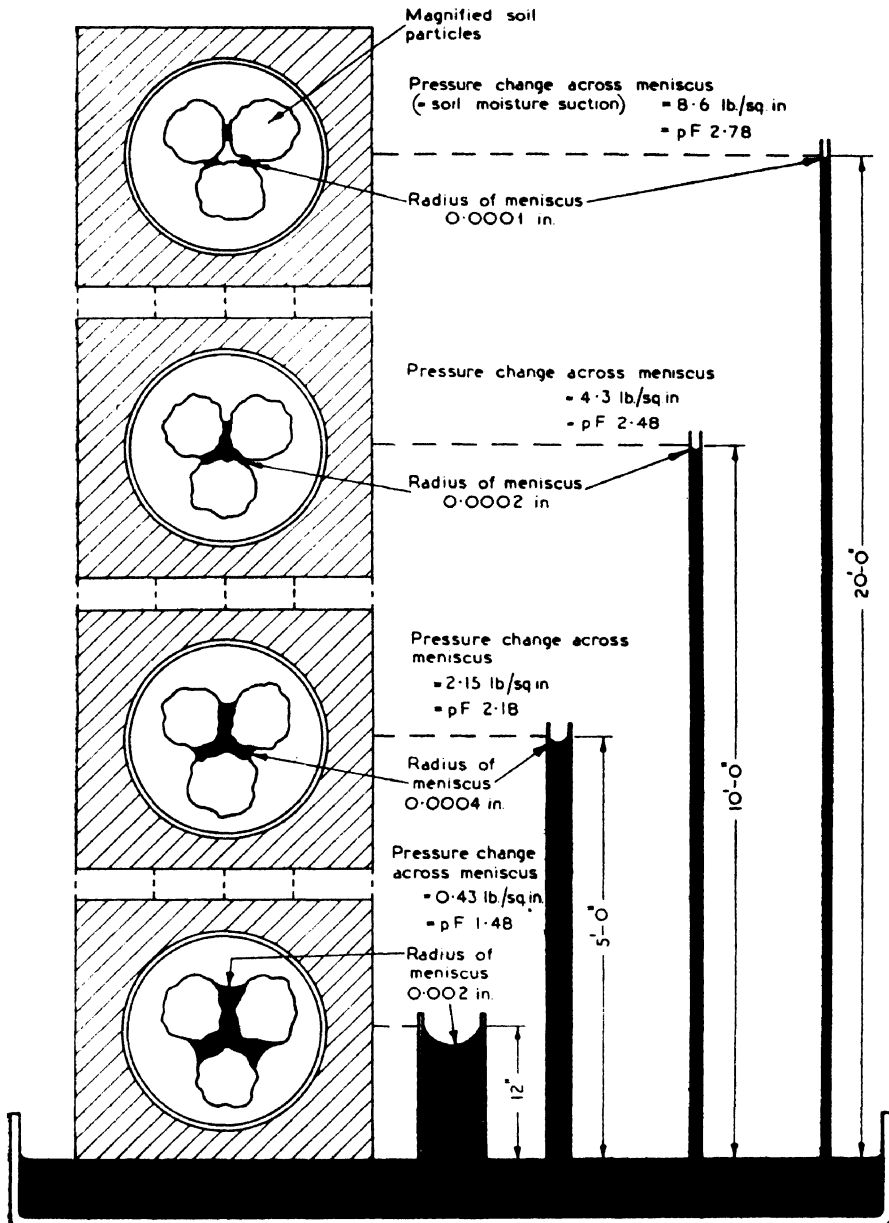



FIG. 16·14 DIAGRAM SHOWING THE CONFIGURATION OF THE AIR-WATER INTERFACES AT DIFFERENT HEIGHTS IN A COLUMN OF UNSATURATED SOIL IN MOISTURE EQUILIBRIUM WITH A WATER-TABLE

surface become equal. This equilibrium pressure is termed the saturated vapour pressure of water at the temperature of the enclosure. If all the water present evaporates before the saturated condition is reached, the final pressure, expressed as a percentage of the saturation pressure, is referred to as the " relative humidity " of the enclosure.

16·39 If moist soil is placed in a similar enclosure, an equilibrium pressure depending on the temperature is again built up. The pressure is however, less than the saturated pressure of water vapour at the same temperature, as a result of the suction with which the soil water is held. The vapour pressure of soil water can therefore be expressed directly, or as a relative humidity (here defined as the ratio of the vapour pressure of the soil to the saturated vapour pressure of water at the same temperature, expressed as a percentage).

16·40 Fig. 16·15 shows the relative humidity/moisture content relationships for a sand and a clay soil[8]. It will be observed that the vapour pressure of the soil increases with moisture content, but that only at low moisture contents, i.e. at high suctions, does the vapour pressure differ considerably from that of free water at the same temperature. Further, the curves show that the vapour pressure of the clay is lower than that of the sand at the same moisture content and temperature.

Vapour Movements under Constant Temperature Conditions

16·41 It follows from Fig. 16·15 that, if two samples of the same soil at different moisture contents are placed together in an evacuated enclosure, evaporation from the wet sample and condensation on the dry sample will

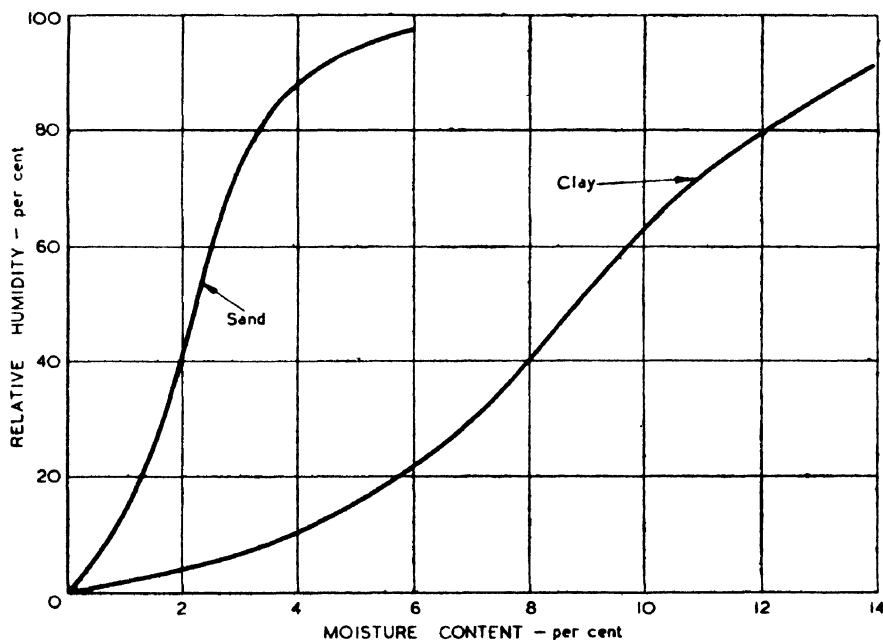



FIG. 16·15 RELATIVE HUMIDITY/MOISTURE CONTENT CURVES FOR A SAND AND A CLAY

occur until both have the same moisture content. On the other hand if the two samples are of different soils, water may actually leave the drier sample to increase the moisture content of the wetter sample if the latter has a higher clay content.

16·42 The obvious similarity between these conclusions and those arrived at from consideration of the soil suction/moisture content curves shown in Fig. 16·11 is indicative of the close thermodynamic relationship between soil suction and vapour pressure, which is discussed in greater detail in the appendix to this chapter. The driving force causing movements of held water under uniform temperature conditions can, in fact, be regarded either as a difference in the suction of the soil or of its vapour pressure. The chief factor which decides whether the movement occurs in the liquid or the vapour phase is the air content of the soil, as will be noted later.

16·43 Vapour pressure equilibria of the type considered above are unaffected by the presence of gases other than water vapour, hence the presence of dry air in the enclosure, previously considered to be evacuated, would have had no effect on the final pressure of the water vapour. In practice air always contains water vapour, the amount depending on the relative humidity of the atmosphere (ratio of the pressure of the water vapour actually in the air to the pressure required to saturate it at the same temperature). Consequently, if moist soil is allowed to come into contact with the atmosphere, water will evaporate from it, or condense in it, until its vapour pressure becomes equal to that of the atmosphere. An oven-dry soil, placed in air to cool, takes up moisture in this way, the first moisture to condense thickening the adsorbed layer on the surface of the soil particles. The moisture content of soil in vapour equilibrium with air is sometimes called the " hygroscopic moisture content". This cannot be regarded as a constant for a particular soil since its value depends on the relative humidity of the atmosphere. In a saturated atmosphere (100 per cent relative humidity) the hygroscopic moisture content may in fact approach saturation.

16·44 In the interior of soil, the relative humidity of the atmosphere has little effect on the vapour pressure, which will depend on the moisture content and temperature of the soil. As the surface is approached, however, the vapour pressure of the air has an increasing effect. If the soil vapour pressure is higher than that of the atmosphere, evaporation occurs which reduces the moisture content of the soil to a progressively increasing depth. Experience during prolonged droughts in this country indicates that the effect of surface evaporation may extend for several feet into the soil. The effect can be observed in Fig. 16·13. If the vapour pressure of the soil is lower than that of the atmosphere, the converse process will occur and water will be transferred to the water-table.

Variation of Vapour Pressure with Temperature

16·45 It was stated above that water placed in an evacuated enclosure evaporates until a certain fixed pressure, depending on the temperature, is created in the enclosure. It is found that an increase in temperature results in further evaporation accompanied by an increase in pressure, whilst a decrease in temperature causes condensation and a consequent decrease in vapour pressure. The vapour pressure/temperature relationship for saturated water vapour is shown in Fig. 16·16.

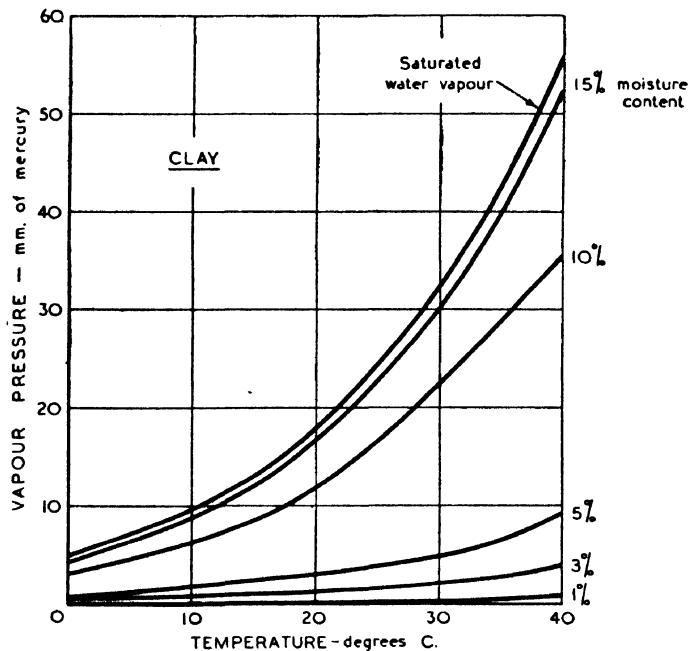
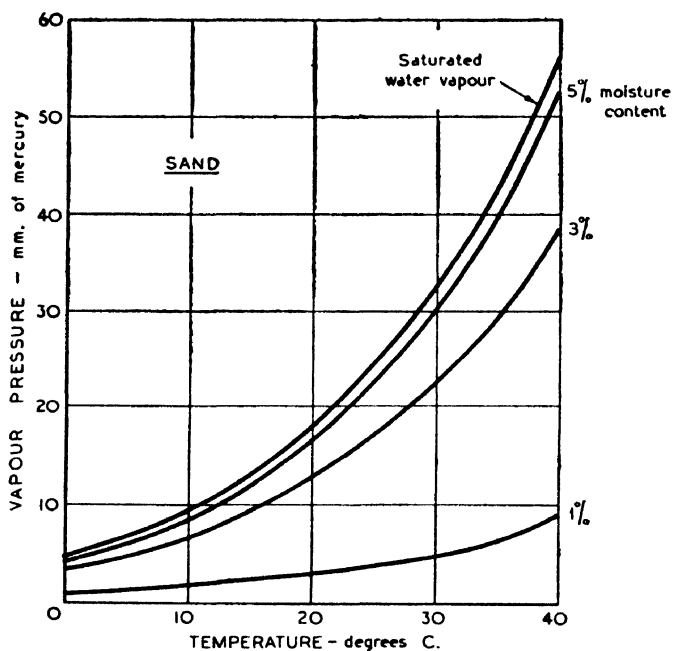



FIG. 16·16 VAPOUR PRESSURE/TEMPERATURE CURVES DEDUCED FROM THE HUMIDITY/MOISTURE CONTENT CURVES SHOWN IN FIG. 16·15

16·46 The vapour pressure of soil water varies with temperature in a similar manner to that of saturated water vapour but the relative humidity of soil water vapour is almost unaffected by temperature. By definition, this means that, if the relative humidity of soil water vapour is x per cent, the vapour pressure is x per cent of the saturated vapour pressure of water at any temperature encountered under practical conditions. It follows that the vapour pressure/temperature relationship for a soil at any moisture content can be obtained if the relative humidity/moisture content relationship for the soil and the vapour pressure/temperature relationship for saturated water vapour, are both available. Fig. 16·16 shows vapour pressure/temperature curves deduced in this manner for the two soils for which relative humidity/moisture content data are given in Fig. 16·15. The curves shown in Fig. 16·16 emphasize that, even in a clay soil, at moisture contents above about 10 per cent, the vapour pressure of the soil water is not materially different from that of free water.

Vapour Movements Associated with Temperature Gradients

16·47 A difference in temperature between neighbouring parts of a soil will give rise to a corresponding difference in vapour pressure, the magnitude of which can be obtained from curves of the type shown in Fig. 16·16. Moisture transferred in the vapour phase to restore the vapour pressure equilibrium evaporates from regions of high temperature (high vapour pressure) and condenses in regions of lower temperature (lower vapour pressure), the movement being accompanied by an exchange of heat (latent heat) tending to reduce the temperature gradient.

16·48 Fig. 16·17 shows some results of laboratory tests carried out at the Road Research Laboratory. Cylindrical samples 11 cm. long, compacted to a known dry density at different moisture contents, were each subjected to a constant temperature gradient of approximately 1·7°C./cm., the sides of the samples being lagged to prevent heat losses. As would be expected from the vapour pressure/temperature relationship, there was a migration of moisture towards the cooler end, which continued for about three days. The distribution of moisture at the end of this period agreed reasonably closely with calculations based on the vapour pressure curves for the soil.

16·49 The following conclusions were reached from a series of tests carried out on soils of different types, dry densities and initial moisture contents:—

(1) Vapour movements occur most readily in soils having low moisture contents; above the plastic limit the movements are very small.

(2) Vapour movements occur less readily in well compacted soils than in loose soils of the same moisture content.

It follows from these conclusions that any factors which reduce the number and size of the free channels through which vapour can pass impede movements of moisture in the vapour phase.

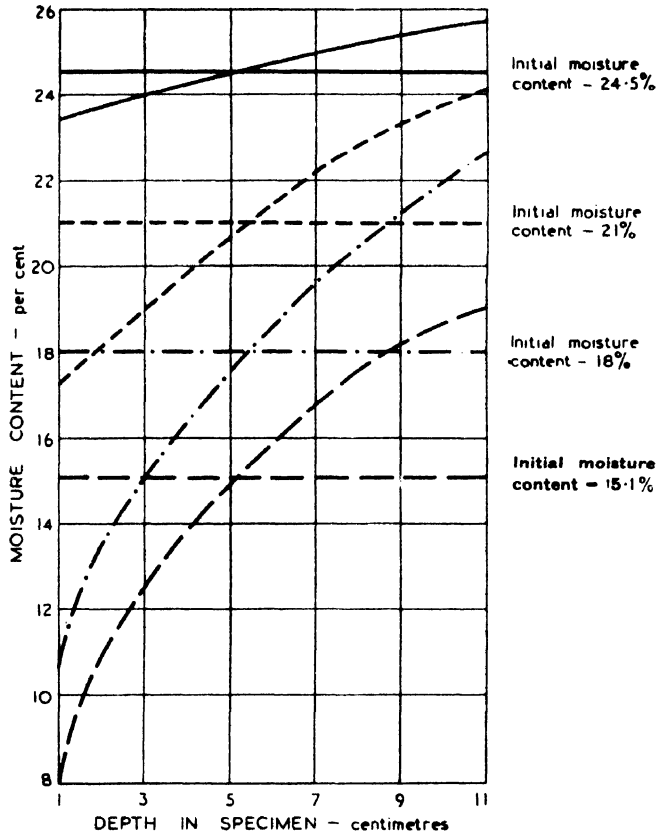
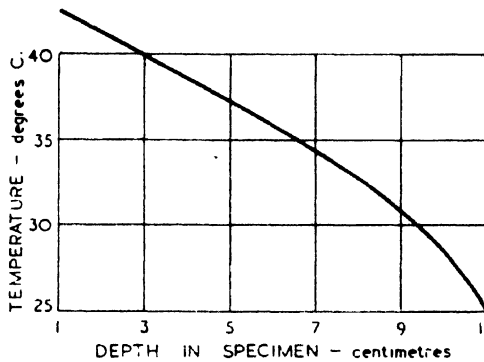
16·50 Movements of water vapour may occur in road subgrades as a result of temperature gradients set up in the earth's surface by the daily and annual atmospheric temperature cycles. The daily cycle, which affects only the top few inches, is unlikely to cause movements of moisture on a large scale. Some condensation of moisture may occur on the under side of road structures during

(a) Equilibrium moisture distribution in specimens
(Dry density of soil 97 lb./cu. ft)

(b) Temperature gradient in specimens

FIG. 16·17 EQUILIBRIUM MOISTURE DISTRIBUTION IN CYLINDRICAL CLAY SPECIMENS OF DIFFERENT INITIAL MOISTURE CONTENTS WHEN SUBJECTED TO THE SAME TEMPERATURE GRADIENTS

the cooling part of the cycle, but theoretical considerations and laboratory tests show that the water of condensation re-evaporates rapidly as the surface temperature increases.

16·51 The annual temperature cycle, however, which sets up temperature gradients which influence the soil to a depth of 30 or more feet, requires further consideration. Fig. 16·18 shows mean monthly temperatures measured in this country at depths of 1, 5 and 9 ft[3]. In summer there is a decrease in temperature of approximately 0·5°C./ft, over the top 10 ft, followed by a reverse gradient in winter.

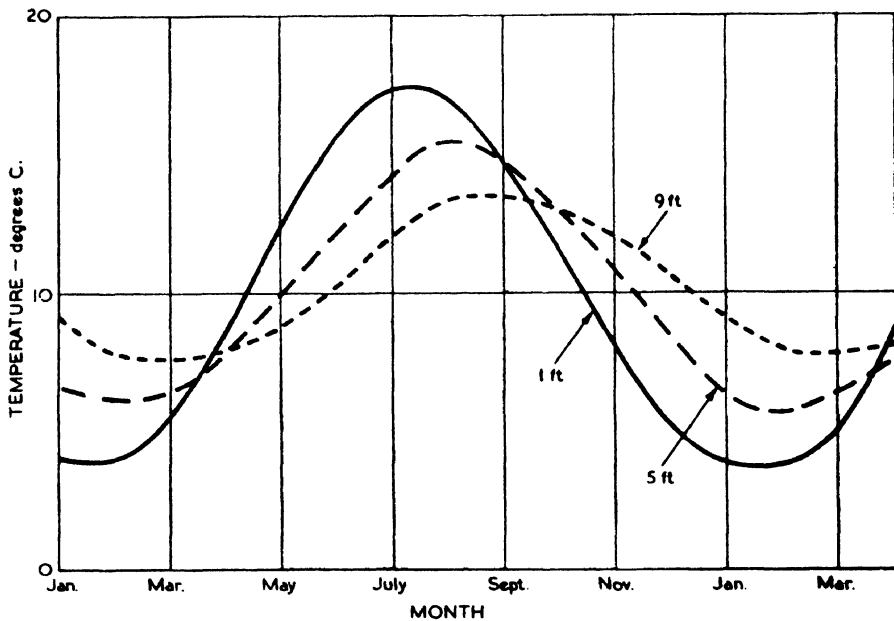



FIG. 16·18 VARIATION OF TEMPERATURE AT VARIOUS DEPTHS DURING ANNUAL TEMPERATURE CYCLE

Observations made at Oxford

16·52 These temperature gradients tend to cause an upward movement of moisture in winter, and a similar downward movement in summer, the magnitude of the movement being predictable from vapour pressure/temperature data for the soil. In this country, where road subgrades normally have a moisture content rather above the plastic limit, moisture movements from this cause are unlikely to be serious, although the moisture content of dry fill used in summer road construction would be rapidly increased by vapour movements in the ensuing winter. Some road failures thought to be due to moisture moving in the vapour phase have been reported from abroad. These cases have been characterized by thin construction, compatible with naturally dry soil conditions, low rainfall and extreme variations of temperature.

Rate at which Movements of Held Water occur

16·53 Little attention has so far been given to the rate at which movements of held water occur. In this connexion distinction must be drawn between the transfer of moisture in the liquid phase at constant temperature, and movements of vapour associated with temperature gradients. In the first case where the driving force can be regarded as a suction gradient, the velocity of flow through a sample of soil can, neglecting the effect of gravity, be expressed by a relationship similar in form to Darcy's law, viz:

$$v = K_u \frac{S_1 - S_2}{x}$$

where S_1 and S_2 are the suctions in two planes separated by a small distance x, between which the flow is measured.

In this equation the value of K_u, the coefficient of unsaturated permeability, is not a constant for a particular soil, but depends on the mean value of S_1 and S_2, i.e. on the average moisture content of the soil through which transfer occurs. Thus the flow between two planes having suctions of 2 and 4 lb./sq.in. will be different from the corresponding flow when the suctions are 12 and 14 lb./sq.in. respectively, despite the fact that the suction gradient is unaltered.

16·54 Laboratory tests carried out in the U.S.A.[4] suggest that in a light clay soil the value of K_u when the average suction of the soil moisture was 3 lb./sq.in. (pF 2·3) was approximately 0·0001 ft/day whilst its value when the average suction was 1 lb./sq.in. was 0·0005 ft/day. In a soil of this type, suctions of 1 to 3 lb./sq.in. would correspond to moisture contents of the order of 20 to 25 per cent. The figures show, therefore, that in the range of interest to the soils engineer the unsaturated permeability increases rapidly as the moisture content increases. They also emphasize the very slow rate at which the transfer of moisture resulting from suction gradients is likely to occur. It is in this extreme slowness that the main danger to road stability from this source lies. In Fig. 16·15 it was seen that except at very low moisture contents the variation of vapour pressure of the soil with moisture content is small. The vapour pressure gradients associated with the suction gradients in soil are for this reason usually small. Where differences of temperature occur in the soil, however, although no appreciable suction gradients are created, comparatively large vapour pressure gradients arise. Under conditions favourable for the transfer of vapour it appears probable, therefore, that movement in the vapour phase due to temperature gradients might occur more rapidly than any movements due solely to differences of suction. It is felt that more research work both in the laboratory and under field conditions will have to be carried out, before anything further can be added concerning the rate of moisture flow in unsaturated soil.

APPENDIX TO CHAPTER 16
SOME ASPECTS OF SOIL THERMODYNAMICS

Relationship between Soil Moisture Suction and Vapour Pressure

16·55 The vapour pressure at a curved liquid/vapour interface is known to depend on the curvature of the interface[5].

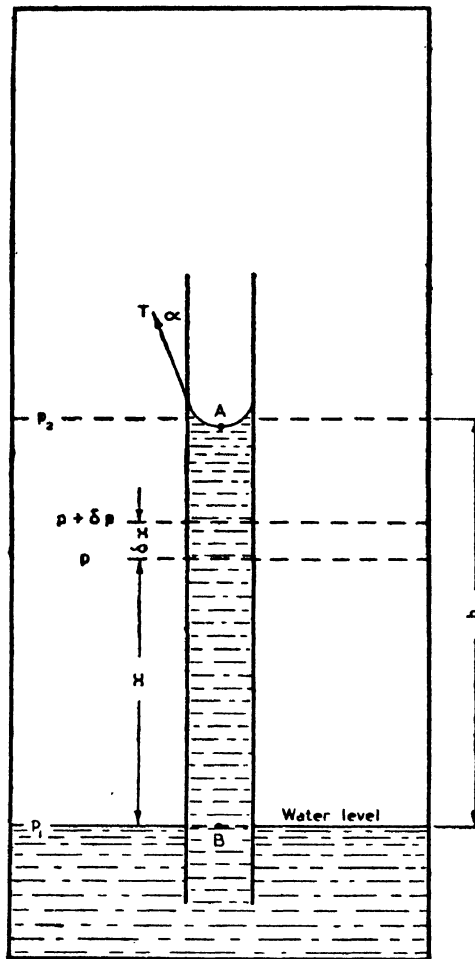



FIG. 16·19 VAPOUR PRESSURE AND SUCTION AT A MENISCUS

16·56 In Fig. 16·19 a vertical narrow-bore tube with its lower end in water is enclosed in an air-tight vessel containing only water vapour above the water surface. Let the height of capillary rise in the tube be h. Consider the variation of pressure with height in the vapour. At height x above the water surface let the pressure in the vapour be p, and the change in pressure corresponding to an increase δx in height be δp. Since the element of vapour contained between the two planes at heights x and x + δ x is in gravitational equilibrium,

$\delta x \, \gamma_v \, g + \delta p = 0$ . . . where γ_v is the density of the vapour at height x and temperature θ

hence, $\frac{dp}{dx} = -\gamma_v \, g$ . . . . (1)

If the vapour behaves as an ideal gas,

$pV = R\theta$ where V is the volume of 1 gram-molecule of the vapour,

or, $\frac{p}{\gamma_v} = r\theta$ where r is the gas constant per gram of the vapour.

Hence, from (1), $\frac{dp}{dx} = -\frac{p}{r\theta} g$

or $\frac{dp}{p} = -\frac{dx}{r\theta} g$

and $\int_{p_1}^{p_2} \frac{dp}{p} = -\int_0^h dx \frac{g}{r\theta}$

where p_1 is the pressure in the vapour at the free water interface and p_2 is the corresponding pressure at height h,

or $\log_e p_2 - \log_e p_1 = \frac{hg}{r\theta}$

and $\log_e \frac{p_2}{p_1} = -\frac{hg}{r\theta}$ . . . . (2)

But p_1 is the saturated vapour pressure of water at absolute temperature θ, hence

$$100 \frac{p_2}{p_1} = \text{relative humidity at height } h = H.$$

From (2)

$$\log_e \frac{H}{100} = -\frac{hg}{r\theta}$$

and since M.r. = R where M is the molecular weight

$\log_e \frac{H}{100} = -\frac{hg}{\theta} \frac{M}{R}$ . . . . (3)

16·57 Applying equation (3) to the case of water, the symbols involved have the following values:—

g = acceleration due to gravity, 981 cm. sec.$^{-2}$

M = molecular weight of water in grams, 18·0 gm mole.$^{-1}$

θ = absolute temperature, 273 + temperature on Centigrade scale.

R = universal gas constant, $8{\cdot}315 \times 10^{+7}$ ergs. mole.$^{-1}$ °C.$^{-1}$

At 20°C.

$$\log_e \frac{H}{100} = -h \times 7{\cdot}25 \times 10^{-7}$$

Equation (3) gives directly the vapour pressure at height h above the water surface, i.e. at the meniscus in the narrow-bore tube, where the corresponding suction is equivalent to a water column of height h. Further, since by Dalton's law of partial pressures, the equilibrium condition of the water vapour considered above would not be sensibly affected by the presence of air, equation (3) will hold to a close degree of approximation even if the vessel considered in Fig. 16·19 is open to the air. The equation can therefore be regarded as giving the relationship between suction and vapour pressure at a water meniscus.

16·58 In soil the suction arises from a large number of water vapour/water menisci of the type already considered. In a state of equilibrium the suction at all these interfaces will be equal, and the equation deduced above, relating suction and vapour pressure, can be applied directly.

16·59 RELATIONSHIP BETWEEN pF AND VAPOUR PRESSURE. The pF value of the suction equivalent to the common logarithm of the suction expressed in centimetres of water, is given by

$$pF = \log_{10} h \text{ in equation (3) if h is expressed in cm. of water.}$$

From equation (3) $h = -\frac{R\theta}{Mg} \cdot \log_e \frac{H}{100}$ . . . . (4)

$$\begin{aligned} \text{Hence } pF &= \log_{10} h = \log_{10}\left[-\frac{R\theta}{Mg} \cdot \log_e \frac{H}{100}\right] \\ &= \log_{10}\left[\frac{R\theta}{Mg} \cdot \log_e \frac{100}{H}\right] \\ &= \log_{10}\left[\frac{R\theta}{Mg}\, 2{\cdot}303 \log_{10} \frac{100}{H}\right] \\ &= \log_{10} 2{\cdot}303\left[\frac{R\theta}{Mg}\,(2 - \log_{10} H)\right], \end{aligned}$$

hence

$$pF = \log_{10}\left(2{\cdot}303\,\frac{R\theta}{Mg}\right) + \log_{10}(2 - \log_{10} H) \quad \text{. . . . .(5)}$$

which gives the relationship between pF and vapour pressure at a fixed temperature. Equations (4) and (5) agree with those derived by Schofield from free energy considerations[1]. Equation (5) reduces to

$$pF = 6{\cdot}502 + \log_{10}(2 - \log_{10} H), \text{ for } \theta = 293°K\ (20°C.),$$

R, M and g having their accepted C.G.S. values. This relationship is shown plotted in Fig. 16·20. The curves show that at suctions lower that pF 3 the relative humidity is very close to 100 per cent. At suctions higher than pF 7, on the other hand, the relative humidity approaches zero very rapidly. The curves illustrate the value of data for the vapour pressure of soil in relation to the measurement of pF in the range 5 to 7, which cannot be conveniently covered by the centrifuge method.

16·60 RELATIONSHIP BETWEEN CAPILLARY POTENTIAL AND VAPOUR PRESSURE. The capillary potential, Ψ, at any point in soil is defined as the work performed against the acting capillary forces in moving unit mass of water to the point,

from a region of zero suction (free water-table). It can be shown that the relationship between Ψ and the quantity h referred to above is given by:—

$$\Psi = -g.h$$

hence from (4)

$$\Psi = \frac{R\theta}{M} \cdot \log_e \frac{H}{100} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

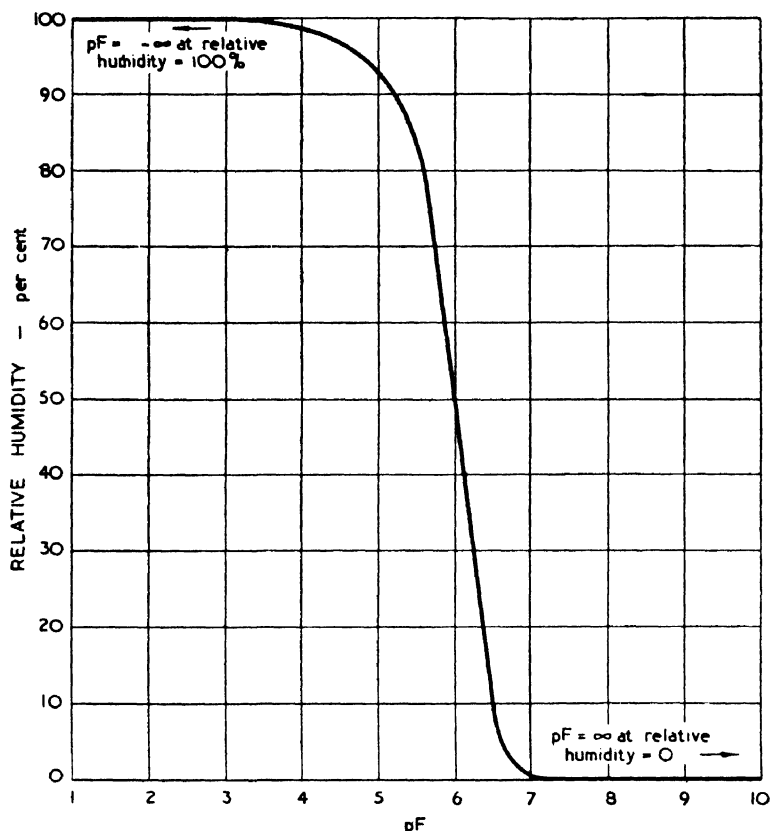



FIG. 16·20 RELATIONSHIP BETWEEN RELATIVE HUMIDITY AND pF OF SOIL AT 20°C.

Variation of Soil Vapour Pressure with Temperature

16·61 Reverting to Fig. 16·19, under equilibrium conditions the difference between the vapour pressure at the plane water surface and at the meniscus in the vertical tube is equal to the sum of the pressure change in the liquid between the points B and A and the change in pressure across the meniscus itself,

i.e., $p_1 - p_2 = \gamma_w gh - \frac{2T \cos \alpha}{a}$ where γ_w is the density of water, T the surface tension, a the radius of the tube and α the contact angle.

hence $\frac{2T \cos \alpha}{a} = \gamma_w gh + p_2 - p_1$

from (2)

$$\log_e \frac{p_2}{p_1} = - \frac{hg}{r\theta}$$

hence

$$\frac{2T \cos \alpha}{a} = p_2 - p_1 - \gamma_w \, r\theta \log_e \frac{p_2}{p_1} \quad \ldots\ldots\ldots (7)$$

Consider the equilibrium at two different temperatures θ_1 and θ_2. At θ_1

$$\frac{2T_{\theta_1} \cos \alpha}{a} = p_{2_{\theta_1}} - p_{1_{\theta_1}} - \gamma_{w_{\theta_1}} \, r\theta_1 \log_e \frac{p_{2_{\theta_1}}}{p_{1_{\theta_1}}}$$

and at θ_2

$$\frac{2T_{\theta_2} \cos \alpha}{a} = p_{2_{\theta_2}} - p_{1_{\theta_2}} - \gamma_{w_{\theta_2}} \, r\theta_2 \log_e \frac{p_{2_{\theta_2}}}{p_{1_{\theta_2}}}$$

$$\text{hence } \frac{T_{\theta_1}}{T_{\theta_2}} = \frac{p_{2_{\theta_1}} - p_{1_{\theta_1}} - \gamma_{w_{\theta_1}} \, r\theta_1 \log_e \frac{p_{2_{\theta_1}}}{p_{1_{\theta_1}}}}{p_{2_{\theta_2}} - p_{1_{\theta_2}} - \gamma_{w_{\theta_2}} \, r\theta_2 \log_e \frac{p_{2_{\theta_2}}}{p_{1_{\theta_2}}}} \quad \ldots\ldots\ldots (8)$$

By reasoning similar to that used in paragraph 16·58, equation (8) can be applied directly to soil, $p_{1_{\theta_1}}$ and $p_{1_{\theta_2}}$ being the saturated vapour pressures of water at temperatures θ_1 and θ_2, and $p_{2_{\theta_1}}$ and $p_{2_{\theta_2}}$ the corresponding vapour pressures of the soil. Since all the quantities in equation (8) with the exception of $p_{2_{\theta_1}}$, $p_{2_{\theta_2}}$, θ_1 and θ_2 are available from tables, the equation gives the variation of vapour pressure with temperature.

16·62 Equation (8) can be conveniently expressed in terms of relative humidity:—

$$\frac{T_{\theta_1}}{T_{\theta_2}} = \frac{p_{1_{\theta_1}} \left(\frac{H_{\theta_1} - 100}{100} \right) - \gamma_{w_{\theta_1}} \, r\theta_1 \log_e \frac{H_{\theta_1}}{100}}{p_{1_{\theta_2}} \left(\frac{H_{\theta_2} - 100}{100} \right) - \gamma_{w_{\theta_2}} \, r\theta_2 \log_e \frac{H_{\theta_2}}{100}} \quad \ldots\ldots\ldots (9)$$

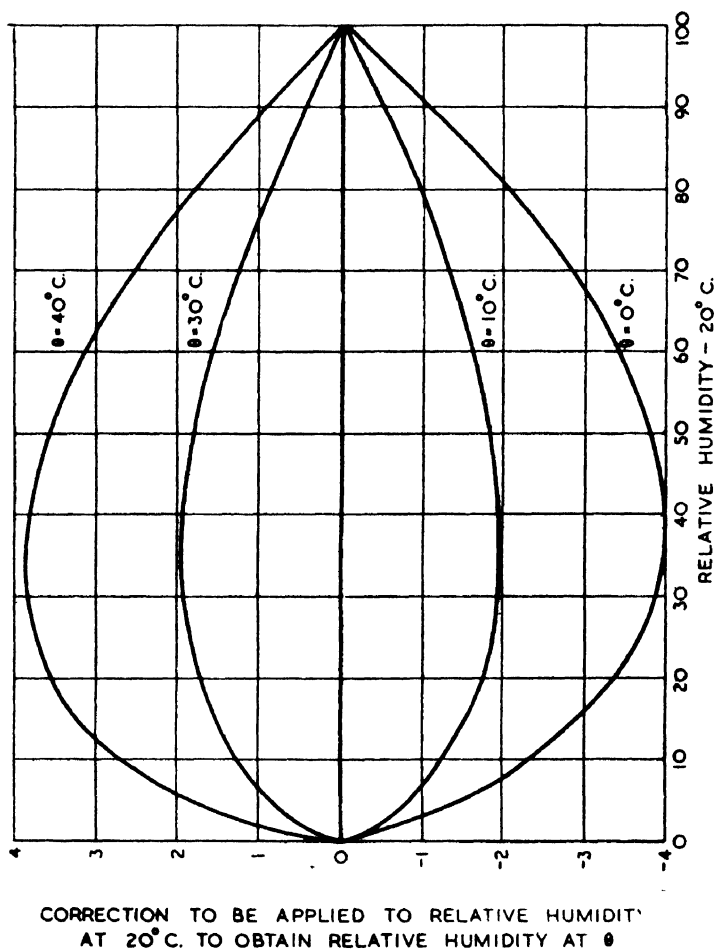



FIG. 16·21 CORRECTION TO BE APPLIED TO RELATIVE HUMIDITY OF SOIL AT 20°C. TO OBTAIN RELATIVE HUMIDITY AT OTHER TEMPERATURES

Examination of this equation shows that the left-hand factors in both numerator and denominator are very small in comparison with the right-hand factors and can be neglected. Hence equation (9) reduces to:—

$$\frac{T_{\theta_1}}{T_{\theta_2}} = \frac{\gamma_{w_{\theta_1}}}{\gamma_{w_{\theta_2}}} \quad \frac{\theta_1}{\theta_2} \quad \frac{\log_{10} \frac{H_{\theta_1}}{100}}{\log_{10} \frac{H_{\theta_2}}{100}} \qquad \ldots\ldots\ldots (10)$$

This equation can be used to obtain the variation of relative humidity of soil with temperature. It can in particular be used to express the relative humidity of soil at other temperatures in terms of the relative humidity at 20°C. This relationship based on equation (10) is plotted in Fig. 16·21 for a range of temperatures between 0° and 40°C.

Variation of pF with Temperature

16·63 From equation (4)

$$h = \text{antilog}_{10}\ \text{pF} = -\ \frac{R\theta}{Mg}\ \log_e \frac{H}{100}$$

$$\text{hence, } \log_e \frac{H}{100} = -\ \left(\text{Antilog}_{10}\ \text{pF}\right) \frac{Mg}{R\theta}$$

$$\text{and } \frac{H}{100} = \text{Antilog}_e \left[-\ \frac{Mg}{R\theta}\ \text{Antilog}_{10}\ \text{pF} \right]$$

Substituting in equation (9), and putting $r = \frac{R}{M}$

$$\frac{T_{\theta_1}}{T_{\theta_2}} = \frac{\gamma_{w_{\theta_1}} g\ \text{Antilog}_{10}\ \text{pF}_{\theta_1} + p_{1_{\theta_1}} \quad \left\{ \left[\text{Antilog}_e - \left(\frac{Mg}{R\theta_1} \text{Antilog}_{10}\ \text{pF}_{\theta_1} \right) \right] - 1 \right\}}{\gamma_{w_{\theta_2}} g\ \text{Antilog}_{10}\ \text{pF}_{\theta_2} + p_{1_{\theta_2}} \quad \left\{ \left[\text{Antilog}_e - \left(\frac{Mg}{R\theta_2} \text{Antilog}_{10}\ \text{pF}_{\theta_2} \right) \right] - 1 \right\}}$$

It follows from equation (9) that the right-hand factors of both numerator and denominator are very small in comparison with the left-hand factors over the whole pF range (0·002 per cent at pF 3), and they can be neglected.

$$\text{Hence, } \frac{\gamma_{w_{\theta_2}}}{\gamma_{w_{\theta_2}}} \quad \frac{T_{\theta_1}}{T_{\theta_2}} = \frac{\text{Antilog}_{10}\ \text{pF}_{\theta}}{\text{Antilog}_{10}\ \text{pF}_{\theta_2}} \qquad \ldots\ldots\ldots (11)$$

$$\text{or } \text{pF}_{\theta_1} - \text{pF}_{\theta_2} = \log_{10} T_{\theta_1} - \log_{10} T_{\theta_2} + \log_{10} \gamma_{w_{\theta_2}} - \log_{10} \gamma_{w_{\theta_1}} \quad \ldots (12)$$

It follows from equation (12) that the variation of pF with temperature is small. In Fig. 16·22 the change in pF from the value at 20°C. for temperatures in the range 0 to 40°C. is shown. The curve can be used to obtain the change in pF between any two temperatures in this range.

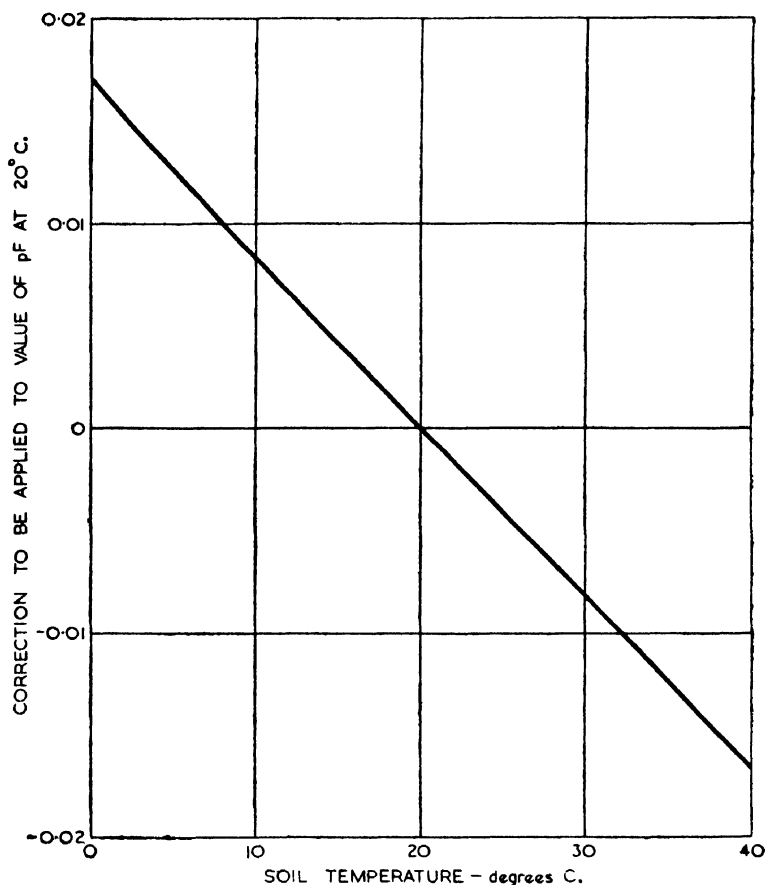



FIG. 16·22 CORRECTION TO BE APPLIED TO pF OF SOIL AT 20°C. TO OBTAIN pF AT OTHER TEMPERATURES

Discussion

16·64 If vapour pressure/moisture content data are available for a soil at a known temperature, the suction/moisture content relationship for the soil at that temperature can be obtained from the equations given in the preceding sections. Further, the variation of vapour pressure and suction with temperature can be deduced.

SUMMARY

16·65 The water in soil can be broadly divided into three categories: ground water beneath the water-table, gravitational water flowing towards the water-table under the action of gravity, and held water retained in the soil principally by surface tension forces.

16·66 Ground water and gravitational water can both be removed from the soil by suitably placed drains. Although held water cannot be drained directly in this manner, it should not be regarded as static. Its movements, to which the greater part of this chapter is devoted, are determined by suction and vapour pressure equilibria.

16·67 The surface tension and absorptive forces by which it is retained reduce the vapour pressure of the held water, and at the same time impart to the water itself a state of reduced pressure or suction, which is found to increase from zero at saturation to values exceeding 1,000 lb./sq.in. in dry soils. If equilibrium moisture conditions in a soil suffer a local disturbance, the suction and vapour pressure gradients created cause a movement of moisture in the liquid and vapour phases tending to re-establish equilibrium. Since the particle-size distribution is an important factor in determining both the soil suction/moisture content and the vapour pressure/moisture content relationships equilibrium will not, in the case of non-uniform soils, correspond to a state of uniform moisture content.

16·68 Since changes of temperature affect the vapour pressure of soil to a much greater extent than its suction, temperature gradients may be accompanied by a movement of moisture in the vapour phase.

16·69 Some consideration is given to the rate at which movements of held water occur in the liquid and vapour phases.

16·70 An appendix is devoted to a mathematical treatment of some aspects of soil thermodynamics of interest to the soils engineer.

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CHAPTER 17

SUBSOIL DRAINAGE AND MOISTURE CONTROL

INTRODUCTION

17·1 It has been appreciated since roads were first constructed that their stability can only be maintained if the soil foundation remains in a relatively dry condition. The Romans, for instance, included drainage ditches in many of the roads which they constructed. The writings of McAdam* show that the need for controlling the moisture content of the road foundation was fully appreciated in the 19th century. The loss in strength of soil consequent upon an increase in moisture content is demonstrated by Fig. 17·1.

CAUSES OF MOISTURE CONTENT CHANGES IN SUBGRADES: RESULTING ROAD DEFECTS

17·2 The principal ways in which changes in moisture content can occur in the subgrade of a road are shown in Fig. 17·2. These are:—

(1) By the seepage of water into the subgrade from higher ground adjacent to the road.

(2) By a rise or fall in the level of the water-table.

(3) By the percolation of water through the surface of the road.

(4) By the transfer of moisture either to or from the soil in the verges as a result of differences in moisture content.

(5) By the transfer of moisture to or from lower soil layers.

(6) By the transfer of water vapour through the soil.

Seepage of Water into the Subgrade from Higher Ground adjacent to the Road

17·3 Seepage flow is common in hilly country where a layer of permeable soil overlies an impermeable stratum. The flow of water may approach the road from any direction and may emerge at the ground surface along a line across the road, revealing itself in the form of springs.

Rise or Fall in the Level of the Water-table

17·4 Seasonal changes in the level of the water-table are usually encountered in flat low-lying areas where little lateral flow occurs. In some coastal areas a rise and fall of the water-table is known to be associated with tidal movements. In such cases the resulting change in the moisture content of the soil depends

*One of McAdam's statements [1] is as follows :—" The roads can never be rendered thus perfectly secure until the following principles be fully understood, admitted and acted upon : namely, that it is the native soil which really supports the weight of the traffic; that whilst it is *preserved in dry state* it will carry any weight without sinking . . . that if water pass through a road and fill the native soil, the road whatever may be its thickness loses support and goes to pieces."

on its capillarity and the average depth of the water-table. It is usually considered that for the maintenance of a stable subgrade the water-table should not rise to a level less than 4 ft below the formation.

Percolation of Water through the Surface of the Road

17·5 In the case of concrete roads water may percolate through joints and cracks which are not adequately sealed, while some types of bituminous surfacing are known to be porous. An example of a concrete road where the lack of maintenance of cracks resulted in the rapid disintegration of slabs, is shown in Plate 17·1A. A case of cracking of concrete slabs adjacent to a badly sealed construction joint is shown in Plate 17·1B. "Mud pumping" was also observed at this site.

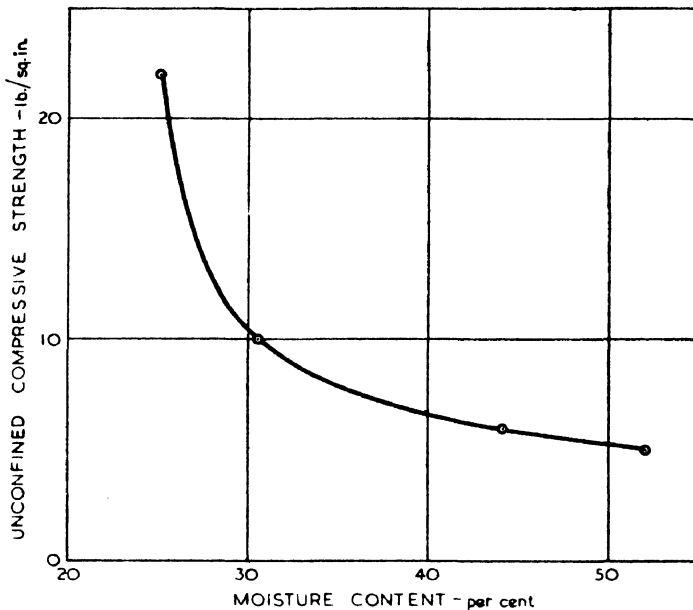



FIG. 17·1 RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND MOISTURE CONTENT FOR A HEAVY CLAY

17·6 Successive freezing and thawing may cause certain types of flexible surface to craze and become porous: this may be soon followed by a complete failure of the road (see Chapter 18).

Transfer of Moisture to or from the Soil in the Verges as a Result of Differences in Moisture Content

17·7 In this country the climatic conditions are such that the moisture content of the soil in the verges of roads is above the average for the year in winter but below in summer. There is a tendency, therefore, for the verges to be wetter than the subgrade in winter but drier in the summer. These differences are likely to cause a transfer of moisture to the subgrade in winter and from the subgrade in summer. In the case of clay subgrades which swell with an increase and shrink with a decrease in moisture content, the edges of the road

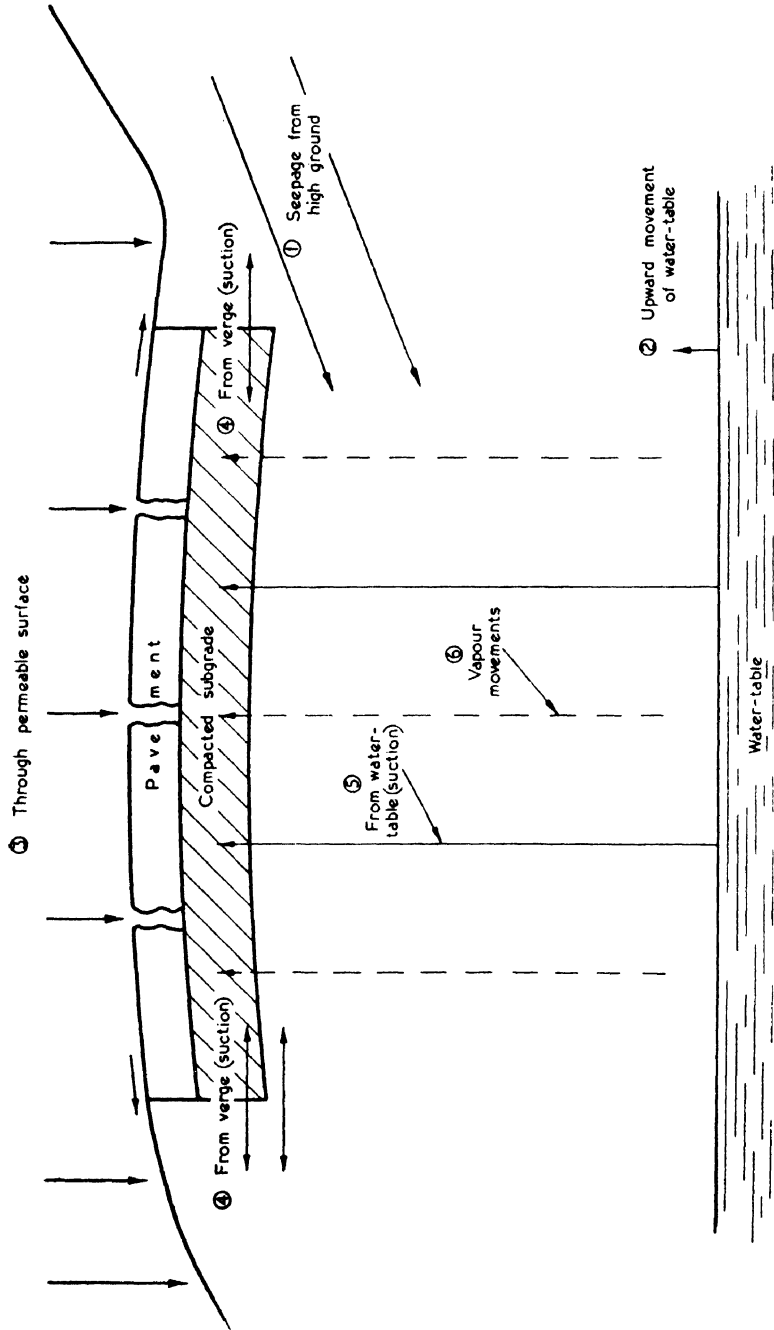



FIG. 17·2 WAYS IN WHICH WATER CAN ENTER AND LEAVE ROAD SUBGRADES

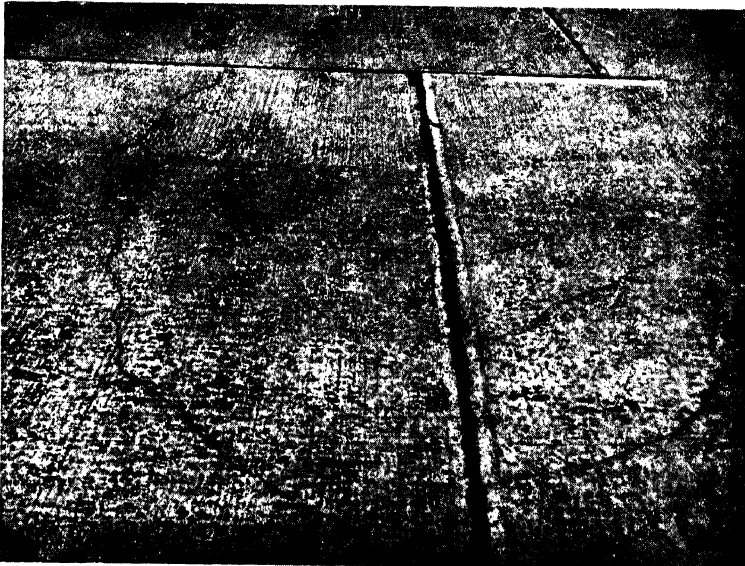
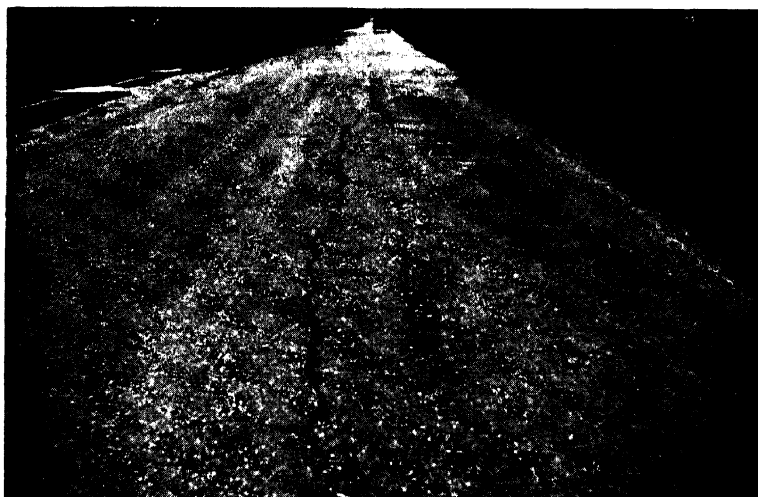
(A) FAILURE OF A CONCRETE ROAD
accelerated by the ingress of water through unsealed cracks

(B) CRACKING OF CONCRETE SLABS
resulting from unsealed construction joints

PLATE 17·1

(A) LONGITUDINAL CRACKING IN A ROAD AFFECTED BY DROUGHT

(B) URBAN ROAD ON A HEAVY CLAY SUBGRADE AFFECTED BY POPLAR AND PLANE TREES

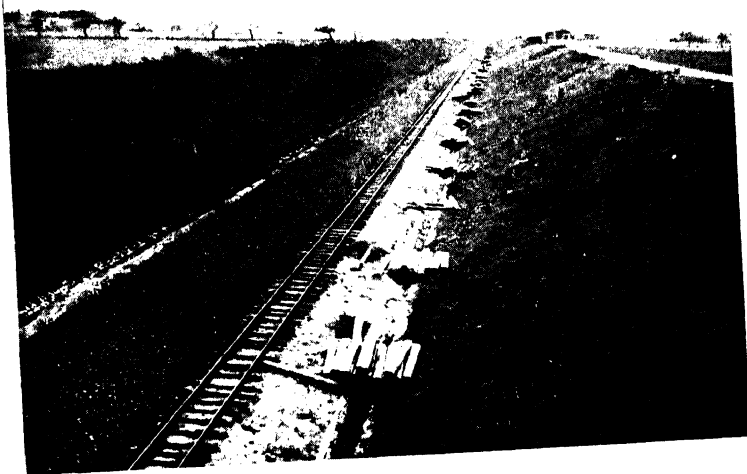
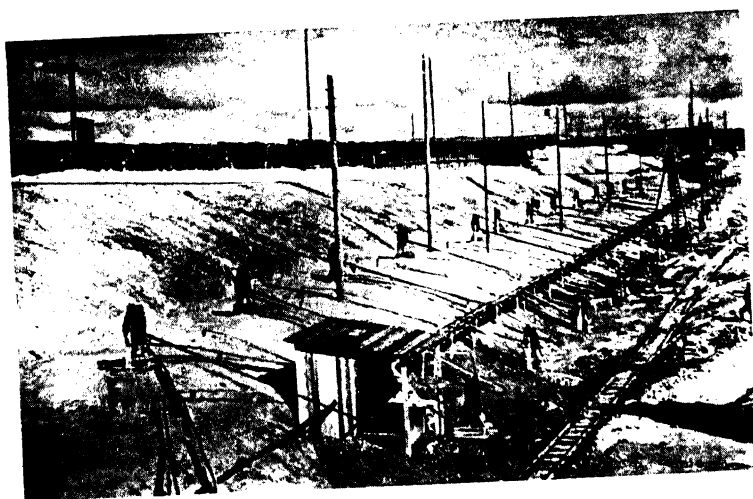



PLATE 17·2

(A) GENERAL VIEW OF CUTTING AT SALZGITTER, LOOKING SOUTH

(B) LAYOUT OF WELL-POINTS ON A SECTION OF THE SITE AT TRONDHJEM

Note the anode between well-points

PLATE 17·3

GENERAL VIEW OF THE CONSTRUCTION WORK FOR THE U-BOAT PENS, TRONDHJEM

The arrows indicate areas of sheet piling buckled by the soil pressure before the installation of the electrical drainage scheme

PLATE 17·4

rise and fall seasonally with respect to the crown. Plate 21·4 shows a concrete road where the slabs were badly displaced by movement of this kind. On this road the edges of the pavement were found to move as much as 2 in. vertically with respect to the crown.

17·8 A particular example of the effect of the transfer of moisture between the subgrade and verges is the damage caused to roads during a prolonged drought such as the one experienced in this country during the summer and autumn of 1947. Many roads of the waterbound macadam type with bituminous surfacings founded on wet clay subgrades, developed severe longitudinal cracking as a result of the shrinkage of the clay under the edges of the roads. A road affected in this way is shown in Plate 17·2A. The drying of the subgrade during the summer may be accelerated by the presence of fast-growing trees, such as poplars, near the road. The effect of trees of this type is most marked in urban areas where a large proportion of the ground is covered with an impermeable surfacing, resulting in a deficiency of water entering the ground from rainfall. Plate 17·2B shows an urban road affected by trees.

Transfer of Moisture to or from Lower Soil Layers

17·9 Depending on the time of construction, the subgrade beneath a road may be wetter or drier than the lower soil layers. Ignoring changes in moisture content due to the causes listed above, the subgrade will tend to reach a state of moisture equilibrium with the lower soil layers.

Transfer of Water Vapour through the Soil

17·10 Vapour movements in soil are associated with differences in vapour pressure which may arise either from differences in moisture content or from differences in temperature existing between different parts of the soil. The transfer of moisture through soil as a vapour can only occur to any large extent when the soil is relatively dry. Clay soils, for example, would have to be at a moisture content well below the plastic limit before any significant movement could occur. Owing to the relatively high moisture content of soils, vapour movements in soil are unlikely to occur in this country to any large extent. There may, however, be a tendency for a subgrade prepared in a dry condition during summer to increase in moisture content during the following winter partly as the result of vapour movements associated with the presence of a downward increase of temperature of approximately ½°C./ft in the top few feet.

17·11 The transfer of vapour through soil is in general likely to be much more pronounced in less temperate climates where large daily and annual fluctuations of temperature are experienced.

17·12 There are thus many ways in which the moisture content of a subgrade can change, some of which can be controlled by conventional methods of drainage but others of which require new methods. Ideally, no change in moisture content of the subgrade should occur during the useful life of the road. The prevention of changes in the moisture content of subgrades thus calls not only for the correct application of traditional drainage methods but also for the use of new techniques which will prevent the transfer of moisture associated with the " held water " in the soil.

FUNCTIONS AND DESIGN OF THE SUBSOIL DRAINAGE SYSTEM

17·13 The road drainage system has to deal with both surface run-off and subsoil water. A detailed discussion of the design of the drainage system to remove surface run-off is outside the scope of this book. However, it has been common practice in the past to employ a combined system in which the surface and subsoil water is collected in a single open-jointed pipe. This may allow the surface run-off to enter the subgrade through the open joints, and is considered to be undesirable, particularly in clay soils. The surface run-off from the road should, in such cases, be carried away through impermeable close-jointed pipes.

17·14 The first step in the design of a subsoil drainage system is to carry out a survey of the soil and moisture conditions in the subgrade.
The following information is of value:—

(1) Soil profile, i.e. the soil type and thickness of the various strata.

(2) Position of the water-table if this occurs reasonably close to formation level (about 6 to 7 ft).

(3) Position of seepage zones if these are found to exist.

17·15 The most appropriate time of the year for carrying out the survey in the British Isles is towards the end of the winter when the water-table is usually at its highest level and the subsoil at its wettest.

The Control of Seepage Flow

17·16 There are two methods of dealing with the condition of seepage flow. If the seepage zone is narrow and within two or three feet of the surface then the usual procedure is to install an intercepting drain just in the impermeable strata underlying the seepage zone as shown in Fig. 17·3. If, however, the seepage zone is wide or the impermeable strata deep, it is in general impracticable to construct the drainage trench sufficiently deep to intercept all the seepage water. In this case, therefore, the intercepting drain is usually located to keep the seepage water about 4 feet below formation level (Fig. 17·4).

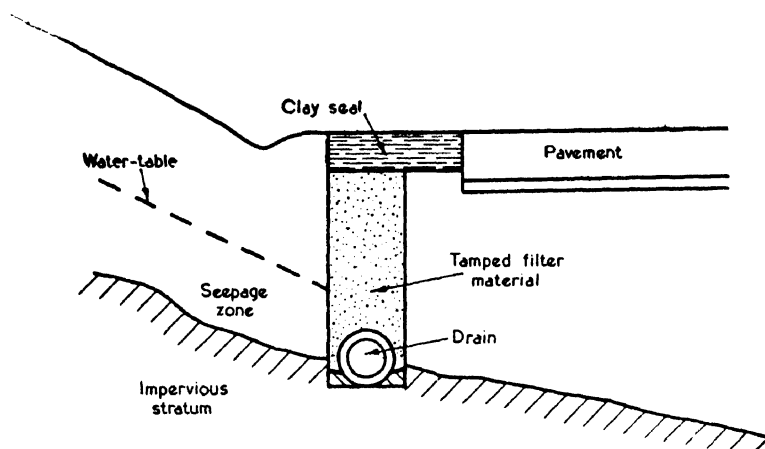



FIG. 17·3 INTERCEPTION OF SHALLOW SEEPAGE ZONE

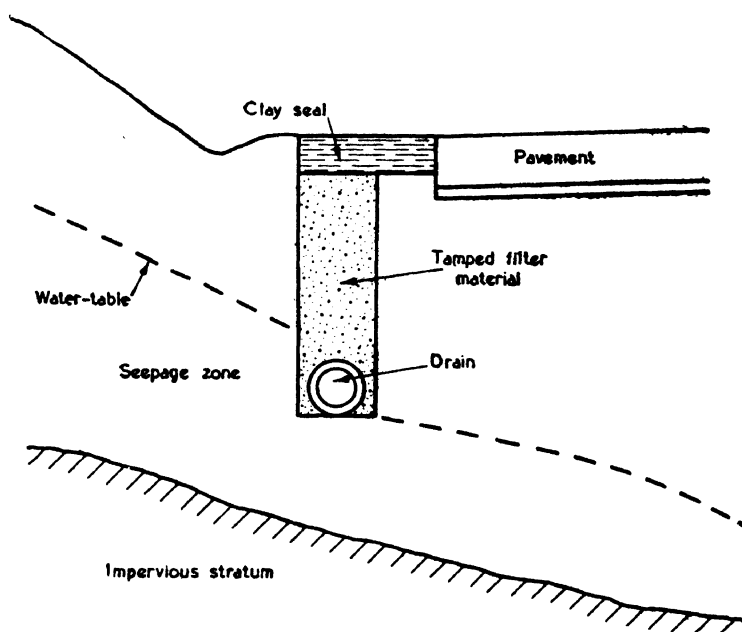



FIG. 17·4 PARTIAL INTERCEPTION OF DEEP SEEPAGE ZONE

17·17 Where roads are on sloping ground longitudinal drains may not be capable of intercepting all the seepage water: in such cases it may be necessary to in tall transverse intercepting drains. Water may sometimes be observed percolating through the surface of a road on or near the bottom of steep hills as a result of seepage from higher ground up the hill.

The Control of a High Water-Table

17·18 A high water-table can be lowered by the installation of a drainage system. It is considered that the water-table should be maintained at a depth not less than 4 feet below formation level (see Fig. 17·5). The actual spacing and depth of drains to achieve this requirement will depend on the soil conditions and the width of the road formation. In the case of dual carriageways, drains may be necessary under the central reservation as well as under the edges of the formation.

17·19 A useful estimate of the effect of installing drains to lower the level of the ground-water at a particular site can be obtained by carrying out a simple field trial. Two parallel trenches, about 50 ft long, are dug on the line of the proposed drainage trenches for the road, to a depth of about two feet below the level to which it is desired to lower the ground water. A transverse line of boreholes at about 5- to 10-ft intervals is sunk between the centre of the trenches and extended about 20 ft on either side. Observations are made of the levels of the water-table in the boreholes before and after pumping the water out of the trenches for a sufficient period of time to establish equilibrium conditions. By plotting these results an estimate can be made of the draw-

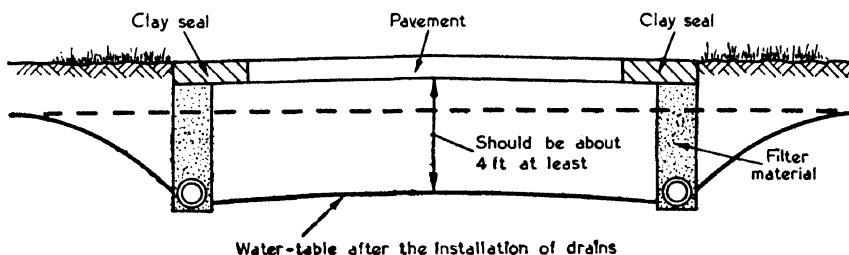



FIG. 17·5 LOWERING THE LEVEL OF THE WATER-TABLE BY THE INSTALLATION OF DRAINS

down effect of the drain trenches, and by this means it is possible to establish the correct depth and spacing of the drains. The capacity required for the drain pipes can be estimated from the rate of pumping necessary to keep the trenches free of water.

17·20 Alternatively, the flow of water to the drains can be calculated from a knowledge of the flow net for the particular soil and drainage conditions, using the coefficient of saturated permeability. The determination of the latter soil constant has been discussed in Chapter 16.

17·21 VERTICAL DRAINS. Where a horizontal impermeable layer of soil overlies more porous strata a perched water-table may exist on top of the impermeable soil. This water may often be drained away by the use of vertical drains. These drains, though of similar form, should not be confused with vertical sand drains which are installed to increase the rate of consolidation of compressible soils. Vertical drains consist of wells cut or bored through the impermeable layer into the porous strata beneath and backfilled with sand or other porous media. The perched water is thus allowed to drain away into the underlying porous strata. This method of drainage has been applied with some success in the case of an airfield in this country.

The Control of Water entering the Subgrade through a Pervious Road Surface

17·22 A completely impermeable road surface is difficult to maintain in practice and a drainage method has been employed to deal with water percolating through the road construction. This method is discussed below together with two other possible ways of dealing with the problem.

17·23 POROUS SUB-BASE. The purpose of the porous sub-base is to trap any water passing through the road surface and lead it to the drain trenches in the verges and so prevent the softening of the subgrade. The porous sub-base consists of 6 to 12 in. of compacted porous material such as sand, gravel, etc., interposed between the pavement and the subgrade (see Fig. 17·6). The subgrade has to be properly cambered and free from depressions and the porous sub-base must connect with the drainage trench. Unless very careful attention is given to the shaping and cambering of the subgrade it is probable that most of the water passing into the porous sub-base would be trapped in irregularities in the surface of the subgrade, and consequently not enter the drain.

17·24 Besides acting as a drain, the porous sub-base increases the thickness of construction and hence reduces the stresses set up in the subgrade by traffic loads. It also prevents soft clay working up into the base of a flexible pavement founded on a clay subgrade and if placed immediately after the preparation of the formation will help to prevent the disturbance of the subgrade by construction traffic.

17·25 It is probable that the improved performance of roads with porous sub-bases is due to the latter factors rather than the possible drainage which the sub-base effects.

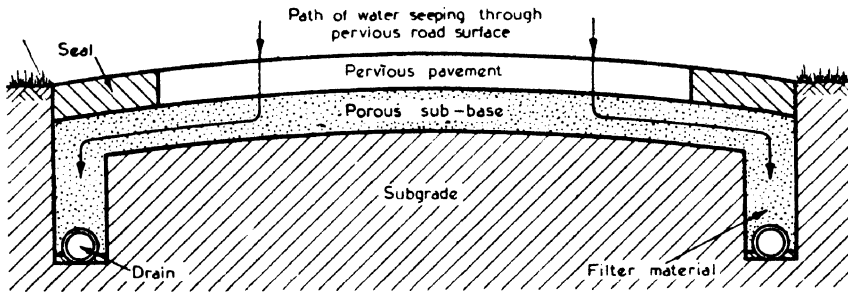



FIG. 17·6 USE OF POROUS SUB-BASE TO REDUCE SUBGRADE DETERIORATION DUE TO PERVIOUS ROAD SURFACE

17·26 STABILIZED SUB-BASE. The stabilization of the top 3 to 6 in. of the subgrade with cement, if practicable, will help to prevent the ingress of water to the subgrade from above. Where the soil type is suitable, the stabilization of a similar depth with either bituminous or other suitable waterproofing agents might also be a satisfactory method, as it will minimize the absorption by the subgrade of water entering through cracks in the surfacing.

17·27 WATERPROOF MEMBRANES. Covering the prepared formation with a waterproof membrane such as prefabricated bituminized surfacing (P.B.S.) has been suggested as a means of preventing surface water softening the subgrade. Provided the membrane does not deteriorate with age this should be a satisfactory method although it might prove expensive. The application of a surface dressing to the formation might also prove a satisfactory method.

Drainage Trench Design

17·28 There are several important features in the design of the drainage trench which contribute to the efficient operation of the drainage system. The trench provides the greater part of the drainage action, the function of the drain pipe being to provide a convenient channel for the rapid removal of the drained water.

17·29 DRAINAGE TRENCH BACKFILLS. In the past little attention has been paid to the nature of the material backfilled into the trench and surrounding the drain pipe, a common practice having been to surround the drain pipe with rubble or beach shingle and to backfill the trench with the excavated material. Sometimes the rubble or shingle surround has been dispensed with

as in normal agricultural drainage practice. It has been found both in this country and in the U.S.A. that this neglect of the backfill has resulted in a very inefficient drainage system which after a few years has ceased to function owing to the silting up of the backfill. In addition, where the drains are installed in silty or sandy soil, fine material is often washed through the rubble surround and carried away with the drainage water. This has led to the formation of large voids (often called "internal erosion") which have caused failure of pavements due to lack of support of the underlying soil.

17·30 The procedure now commonly specified in the U.S.A. is to use specially selected filter material in the drain trench. This filter material is designed to offer little resistance to the flow of water to the drain but to resist the passage of silt and thus prevent the clogging or internal erosion of the backfill.

17·31 DESIGN OF FILTER MATERIAL. Much research into filter materials has been carried out since Terzaghi's original work in 1920. The U.S. Corps of Engineers have developed a design method[2] which has proved quite satisfactory in practice.

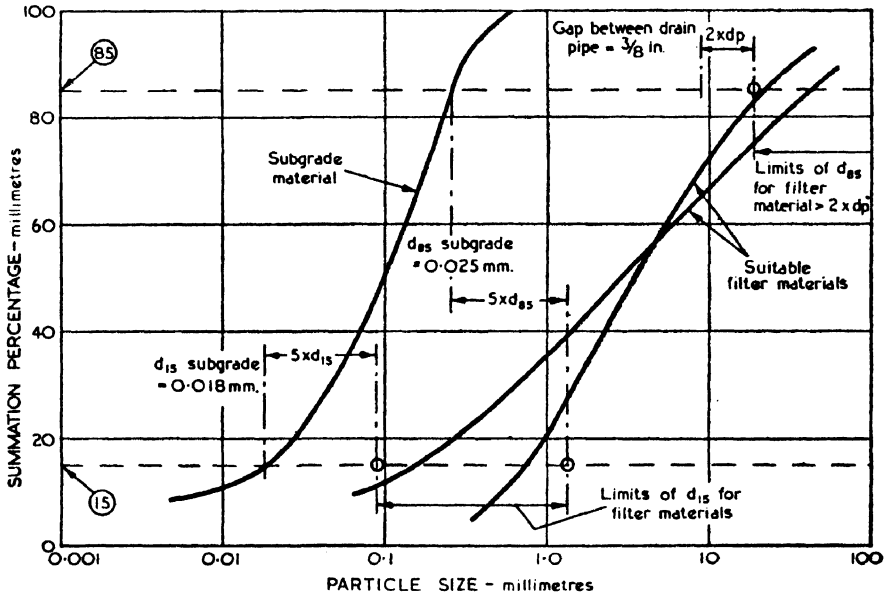



FIG. 17·7 METHOD OF DESIGN FOR FILTER MATERIALS (U.S. Corps of Engineers)

17·32 The first step in the design of filter material is to obtain a particle-size analysis of the subgrade soil in which it is proposed to install the drain and to plot a curve of particle-size distribution in the usual manner. The limits for the particle-size distribution of the filter material are based on the following requirements (see Fig. 17·7):—

$$\text{Piping ratio, defined as } \frac{\text{15-per cent size of the filter material}}{\text{85-per cent size of the subgrade material}} \text{ must be} < 5$$

$$\text{Permeability ratio, defined as } \frac{\text{15-per cent size of the filter material}}{\text{15-per cent size of the subgrade material}} \text{ must be} > 5$$

(By 15-per cent size is meant that size of particle corresponding to the 15-per cent ordinate of the chart of particle-size distribution and similarly for the 85-per cent size.)

17·33 The requirement that the piping ratio must be less than 5 is designed to prevent silt or fine particles from the subgrade soil being washed into the filter material, while the requirement that the permeability ratio is greater than 5 is designed to ensure that the filter material will be sufficiently permeable.

17·34 Based on the above two requirements, the limits of the sizes for the 15 per cent size of the filter material should lie between:—

5 x 15 per cent size of the subgrade material and

5 x 85 per cent size of the subgrade material.

(See Fig. 17·7.)

In the case of cohesive subgrades, the 15-per cent size of the filter material need not be less than 0·1 mm.

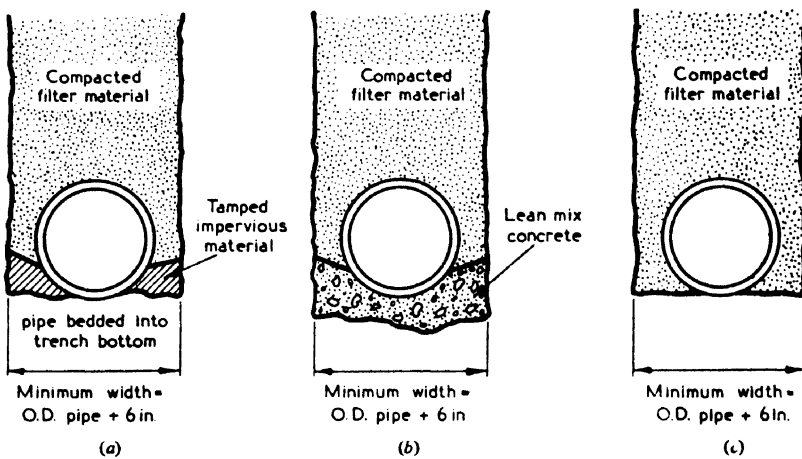
17·35 The limit for the coarse particles of the filter material is based on the size of holes in the pipes (for perforated pipes) or the gap at the joints in the case of open-jointed pipes. The 85-per cent size of the filter material must be greater than twice the size of this gap. In the case of porous concrete pipes, this requirement is unnecessary.

17·36 The backfill should consist of naturally occurring material, but washed sand or crushed stone having a smooth particle-size distribution curve may be used.

17·37 If the particle-size distribution of the subgrade soil and the gap between the drain pipes are such that it is not practicable to find one filter material which will meet all the requirements, then it may be necessary to employ two filter materials, a coarse material placed around the pipe and a finer one between the coarse filter material and the subgrade soil.

17·38 METHOD OF INSTALLATION OF DRAIN PIPES IN TRENCH BOTTOM. The generally recommended method of placing the drain pipes in the bottom of the trench is shown in Fig. 17·8. When the trench is cut into an impermeable soil, as in the case of an intercepting drain, the drain pipe should be bedded into the trench bottom and some of the soil tamped around the lower part of the pipe. Alternatively, the pipe can be bedded into a weak concrete and the lower third of the pipe cemented at the joints. Both methods of installing the drain pipe ensure that the bottom of the trench is kept drained and that there is no possibility of water accumulating under the pipe and subsequently causing deterioration of the subgrade.

17·39 If the soil is permeable and the only function of the drain is to lower the level of the ground-water, then the above precaution need not be taken.

17·40 SEALING THE TOP OF THE DRAIN TRENCH. It is a general practice, when using filter materials, to seal the top of the drainage trench, as the surface runoff usually contains a fair amount of silt in suspension which would tend to cause silting up of the backfill if allowed to enter the trench at the top. A 6-in. thick layer of compacted clay or other similar soil is probably the cheapest method of effecting the seal.

(a) & (b) IMPERVIOUS TRENCH BOTTOM

It is advisable in the case of impervious soils to remove completely all the water entering the drain trench from above. Alternative methods are shown using tamped impervious material (i.e. clay) or lean mix concrete around lower third of the pipe.

(c) PERVIOUS TRENCH BOTTOM

In the case of pervious soils where the action of the drain is to lower the level of the water-table it is unnecessary to carry out an elaborate treatment of the trench bottom.

FIG. 17·8 THE PREPARATION OF DRAIN TRENCHES AND THE INSTALLATION OF PIPES

The Control of Subgrade Moisture Movements arising from Differences in Soil Suction and Vapour Pressure

17·41 Movements of moisture arising from differences in soil suction and vapour pressure cannot be controlled directly by the drainage methods discussed previously and each case requires special treatment. Possible methods of control are discussed below.

17·42 MOISTURE MOVEMENTS ARISING FROM SUCTION GRADIENTS BETWEEN THE SUBGRADE AND THE SOIL BENEATH. The use of a thin horizontal layer of gravel beneath the compacted subgrade is sometimes recommended as a method of intercepting capillary moisture in fine-grained soils. On the basis of present knowledge of the movement of moisture in unsaturated soil, it is considered that this method is unlikely to be effective, the moisture distribution in the soil being unaffected by the interposed layer, although the latter may have a much lower moisture content than the surrounding soil when equilibrium conditions are reached. It is possible, however, that the interposed layer might be effective if a very coarse material having a negligible suction were used.

17·43 Moisture from wetter underlying soil can be intercepted by the use of a horizontal impermeable membrane such as prefabricated bituminized surfacing (P.B.S.) located beneath the compacted subgrade as shown in Fig. 17·9.

17·44 MOISTURE MOVEMENTS TO AND FROM THE VERGES ASSOCIATED WITH SOIL SUCTION DIFFERENCES. Horizontal movements of moisture between the verges and the subgrade can be dealt with in a similar manner to the vertical move-

ments referred to in the previous section. Fig. 17·9 shows how P.B.S. can be used to seal the subgrade completely from the ingress of moisture from any direction.

Not to scale

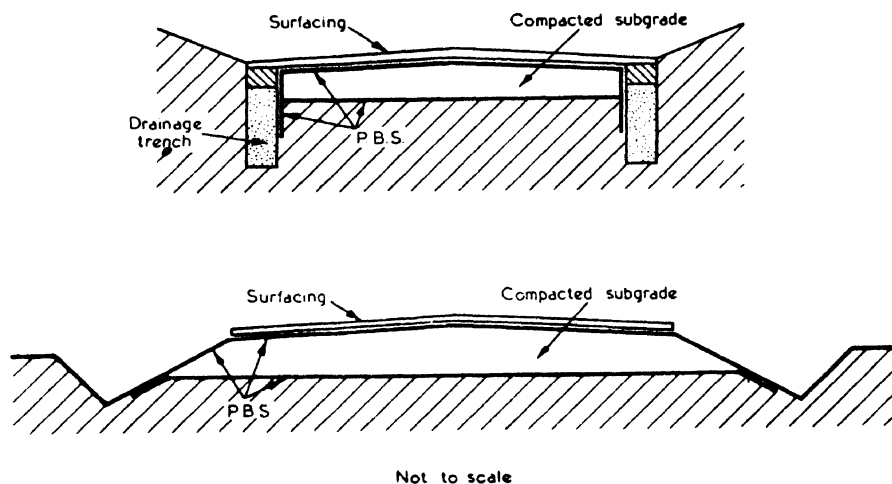



FIG. 17·9 USE OF P.B.S. TO PREVENT ENTRY OF WATER INTO COMPACTED SUBGRADE

17·45 Changes in the moisture content of the subgrade due to this particular cause can be reduced, and possibly eliminated, by providing an impermeable surface for the verge. Stabilized soil verges constructed on some roads in the U.S.A. serve this purpose.

17·46 MOISTURE MOVEMENTS RESULTING FROM VAPOUR PRESSURE DIFFERENCES IN THE SOIL. The use of completely impermeable blanket layers, e.g. P.B.S., similar to those discussed in the previous sections is thought to prevent vapour movements. Such layers have been used in hot dry climates where road and airfield failures have occurred which have been attributed to an increase in the moisture content of the subgrade as the result of a transfer of water vapour.

DRAINAGE OF FINE-GRAINED SOILS USING ELECTRO-OSMOSIS

17·47 One of the methods frequently employed in the execution of foundation work in water-logged ground is to lower the ground-water level below the depth of excavation by means of well-points established around the perimeter of the site. However, this method of dewatering foundations can be used only on a fairly limited range of soils. As the proportion of silt and clay in the soil increases, the permeability decreases until a point is reached where the flow to the wells is so slow that their effect is purely local. Glossop and Skempton[3] indicate that ground-water lowering cannot be used in soils having a combined clay and silt fraction exceeding about 60 per cent of which more than about 10 per cent is clay. There is, therefore, a definite limitation to the use of well-points in ground-water lowering.

17·48 During the war, however, Dr. Leo Casagrande[4] made use of the principle of electro-osmosis in excavation work in fine-grained soils. The electro-osmotic effect with soils has been known for a considerable period of time but the German engineers were the first to apply it to the control of ground-water.

Theory of Electro-osmosis

17·49 The double layer theory developed by Freundlich[5] can be used to explain electro-osmosis in soil. The assumption is made that the water adjacent to the soil particles consists of two layers of molecular dimensions, one bonded to the soil and the other free to move. In general the layer nearest the soil is characterized by an excess of anions and the other by an excess of cations. Hence, when an external E.M.F. is applied, the movable cations migrate towards the negative electrode.

Electro-osmosis as applied to Ground-water Lowering

17·50 In the application to the control of ground-water, the metal lining of the well or well-point is made the negative electrode (cathode) and a suitable conductor such as a steel tube is sunk into the ground between the well-points to act as the positive electrode (anode). When an electrical potential is applied between the cathode and anode, moisture will flow towards the cathode from the surrounding soil and can be pumped away in the usual manner. In practice the electrodes are usually arranged so that the electro-osmotic forces oppose the hydraulic gradient, and thus prevent the flow of water which is often the cause of instability during excavation work. In such cases the method has been shown to be effective although large quantities of water are not removed from the soil.

17·51 It must be stressed that electro-osmosis has been applied only in a limited number of cases, and although it appears to have many possibilities, the conditions necessary for its successful application may not occur very frequently.

17·52 During the war there were several full-scale applications[4] of the method in German civil engineering work and two of these projects are described in an appendix to this chapter.

APPENDIX TO CHAPTER 17

EXAMPLES OF THE FULL-SCALE APPLICATION OF ELECTRO-OSMOSIS

Salzgitter Cutting

17·53 This cutting, the location of which was decided by mining considerations and its depth by the level of existing roads which cross it, is situated immediately north of Salzgitter near Brunswick. The cutting was for a double-track railway and was about 1¼ miles long by about 20 ft deep.

17·54 The soil conditions consisted of 4 ft of sandy soil on a very soft silt, the latter being so soft as to be impossible to excavate with mechanical shovels. It was also impossible to form stable slopes in any excavation.

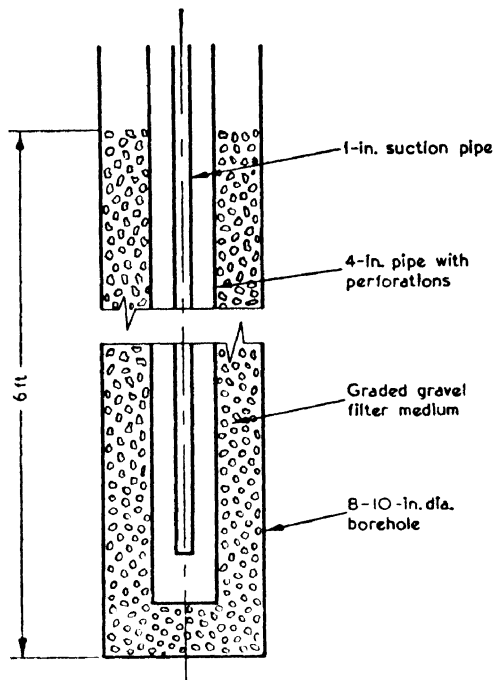



FIG. 17·10 SECTION THROUGH FOOT OF WELL-POINT

17·55 Two lines of well-points were sunk to a depth of about 25 ft on either side of the proposed cutting, the spacing between the well-points being about 33 ft. A line of 1-in. diameter gas tubing was sunk between the well-points to the same depth to act as anodes. A length of about 330 ft of the cutting was stabilized at a time. The well-points were bored with a 10-in. diameter soil auger and consisted of a 4-in. diameter perforated pipe inserted in the hole and the outside filled for 6 ft from the bottom of the hole with graded gravel-sand filter material (see Fig. 17·10). A plain 1-in. pipe was placed inside the filter pipe for the purpose of pumping water from the well-point. One small centrifugal pump was connected to every two well-points.

17·56 Initially, a potential difference of 180 volts was applied between the electrodes and the current consumption amounted to about 19 amps. per well-point. To reduce the consumption of electrical energy the potential difference was reduced to 90 volts when the floor of the cutting was reached. The well-points were normally pumped out at intervals, the flow being insufficient to require the pumps to be run continuously. Without the application of the current the flow from the 20 well-points was about 90 gall./24 hours. After the potential was applied the rate of flow rose to about 13,500 gall./24 hours; an increase of 150-fold. The filter material in each well-point silted up after about 3 weeks and a new point had to be sunk. The electrical drainage stabilized the silty soil sufficiently to enable a face shovel to be used to excavate it to side slopes of 1:1.

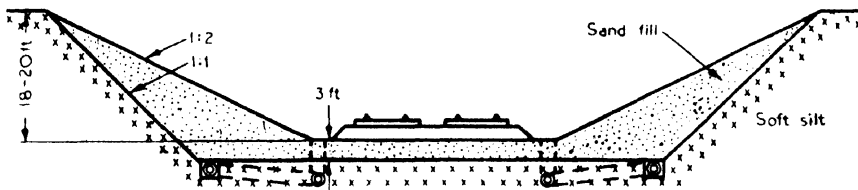



FIG. 17·11 CROSS-SECTION THROUGH CUTTING AT SALZGITTER

17·57 After the completion of the excavation the slopes and floor of the cutting were covered with sand (see Fig. 17·11) to prevent the flow of the soil after the current was switched off. Plate 17·3A shows a view of the finished cutting. The cost of the electrical drainage scheme amounted to about sixpence per cubic yard of excavation.

U-Boat Pens, Trondhjem

17·58 The construction of the U-boat pens at Trondhjem involved the excavation of an area about 700 ft by 500 ft to a depth of about 45 ft through variable soft silt with sand veins. During the initial period of construction by traditional methods, considerable difficulties were met and sheet piling suffered serious damage due to buckling. In addition, a point was reached at which the silt flowed into the excavated area as fast as it could be removed. As a last resort it was decided to use the electrical drainage method which had been applied successfully at Salzgitter.

17·59 The electrical drainage process was intended to produce a stable belt of soil around the whole perimeter of the site. An inner and outer ring of sheet piling was driven around the entire site and a double row of well-points were sunk between these rings of sheeting. The two rows were about 45 ft apart, and in each row, the wells were spaced at 30-ft intervals with 1½-in. gas piping intermediately between them to serve as anodes. The wells were similar to those employed at Salzgitter.

17·60 Each well was furnished with a small pump and the delivery pipe was connected to a collecting main and measuring tank. Plate 17·3B shows some of the well-points and their layout.

17·61 Power was supplied from a rotary converter capable of an output of 2,000 amps. Current consumption averaged 20-30 amps. per well at a potential difference of 40 volts. The application of the current increased the flow of water to the wells by about 10 times. On an average a total flow of 28,000 gall. of water per day was obtained in dry weather. With the incidence of autumn rains, this rose to 47,000 galls. per day.

17·62 The system was kept in operation until the excavation and floor concreting had been completed, no trouble being experienced from the previously unstable soil conditions. Plate 17·4 shows a general view of the constructional work.

17·63 The cost of the electrical energy supplied to stabilize the soil amounted to about 0·14 pence/cu.yd.

SUMMARY

17·64 The control of the moisture content of the subgrade is an essential feature of a good road. Water moving freely under the action of gravity can affect the moisture content of a subgrade by reaching it through a pervious road surface, by seepage and by a rise in level of the water-table. Moisture can also enter or leave the subgrade, either as a liquid or as a vapour, under the action of forces entirely inherent in the soil itself.

17·65 Before designing the subsoil drainage system for a road, a survey should be made of the soil and water conditions in the subsoil. Free water can be intercepted or controlled and a high water-table can be lowered by the installation of subsoil drains. The position of the drains should be such that the water-table is maintained at least 4 ft below formation level. The backfill used in the drainage trench should consist of a properly designed filter material, especially where the subsoil is liable to cause silting of the backfill. A procedure for designing filter material is included in the chapter.

17·66 Existing and suggested methods of controlling the transfer of moisture in unsaturated soil are reviewed in some detail.

17·67 The very slow rate of flow of free water through certain types of silty soil can be increased by the application of an electrical potential. The theory of this electro-osmotic process is explained briefly and some applications of the process to full-scale construction work carried out in Europe are described in an appendix.

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CHAPTER 18

FROST DAMAGE TO ROAD FOUNDATIONS

INTRODUCTION

18·1 Damage to roads by frost may be confined to the surfacing, but under severe conditions the base and subgrade may also be involved. The failure of subgrades during periods of extreme cold can be caused by frost heave, that is, the condition where the segregation of ice occurs in the upper layers of the soil. Water entering the subgrade through a surface rendered porous (or more porous) by frost action is also responsible for some failures.

FROST HEAVE

18·2 The theory of frost heave has been discussed by Taber[1], Beskow[2] and others. In certain circumstances, the suction force associated with the freezing process in the soil pores causes an upward migration of water into the freezing zone. This water freezes in horizontal layers or lenses varying in thickness from 1/32 in. to one or more inches. As the ice lenses form, the surface is displaced by an amount approximately equal to the total thickness of the lenses.

18·3 Frost heave can be produced in the laboratory by freezing prepared soil samples. The specimens are frozen from the surface whilst their lower ends are in contact with water or wet soil maintained at a temperature slightly above freezing point. This provides a convenient way of comparing the frost susceptibility of different soils. Plate 18·1 shows a sample of compacted brickearth before and after being subjected to the freezing test.

18·4 In the case of road subgrades the soil normally thaws from the upper layers downwards with the result that a considerable quantity of water may be trapped in the top few inches of soil until released by the complete thaw of the frozen zone. If traffic is allowed to use a road whilst the moisture content of the upper soil layers is abnormally high, the reduced bearing power of the soil may result in serious damage to the surface of the kind shown in Plate 18·2. Damage of this type, where the saturated subgrade is forced to the surface by traffic, is sometimes referred to as frost boils.

Frost Heave in relation to Particle-size Distribution of Soil

18·5 Where ice segregation occurs, the water necessary to supply the growing ice lenses enters the frozen zone through the smaller soil pores which, by virtue of their size, remain ice-free at temperatures rather below 0°C.(32°F.). If the particle-size distribution of a soil is of such a nature that sufficient of these small pores are not available, the water in the soil freezes *in situ* unaccompanied by any migration of moisture. The expansion of the soil water on freezing takes place chiefly into the voids and there is no appreciable change in volume. Coarse sands and gravels fall into this category. Experimental evidence suggests that under the climatic conditions of the British Isles there must be at least

10 to 15 per cent by weight of material finer than 0·02 mm. for the soil to be seriously susceptible to frost. On the other hand, if the soil has a preponderance of fine material the permeability may be too low to allow water to enter the frozen zone at the rate necessary to promote the segregation of the ice. This is the case with heavy and medium clays. In this country soils with a greater percentage of clay (particles smaller than 0·002 mm.) than 30 per cent are unlikely to be affected by frost heave. It follows that heave is most likely to occur in silts and silty sands. Fig. 18·1 shows the limits between which the particle-size curves of frost-susceptible soils are likely to lie. These limits are based on experimental investigations carried out in southern England during periods of frost in the last ten years.

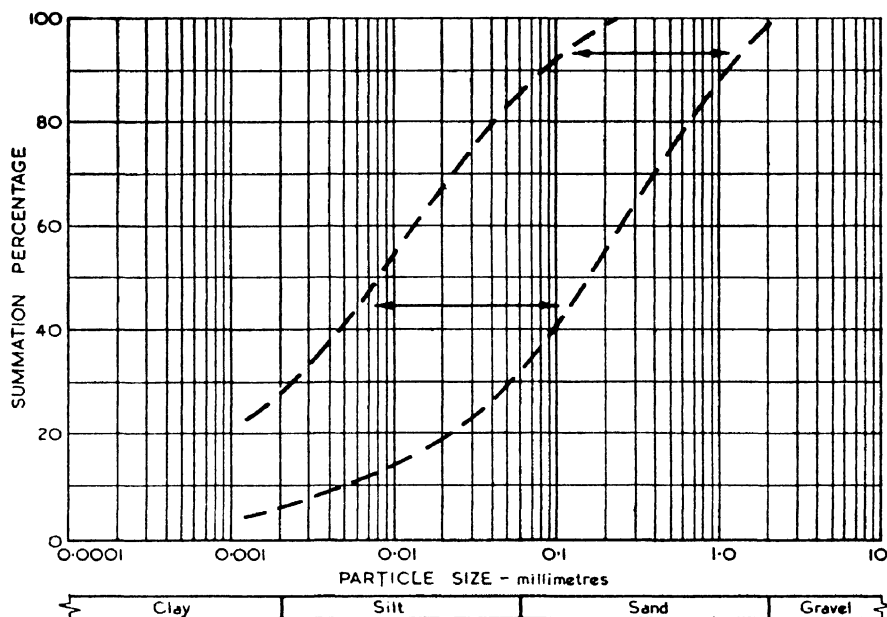



FIG. 18·1 PARTICLE-SIZE LIMITS WITHIN WHICH SOILS ARE LIKELY TO BE FROST-SUSCEPTIBLE

(Based on failure investigations carried out in southern England)

18·6 Some forms of chalk are very susceptible to frost heave, particularly where the natural material has been broken up and re-compacted, e.g. in the case of embankments. The factors affecting the frost-susceptibility of chalk are dealt with in Chapter 7.

Effect of Water-table Level on Frost Heave

18·7 For ice segregation to occur, the suction exerted by the growing ice lenses must be greater than the soil moisture suction in the soil beneath the frozen zone from which the water is drawn. A high water-table, by reducing the suction in the soil, encourages the formation of ice lenses in frost-susceptible soils. Where the water-table is within two feet of the formation level, the lowering of the water-table may reduce materially the liability of a road to frost damage.

Effect of Thickness of Construction on Frost Heave

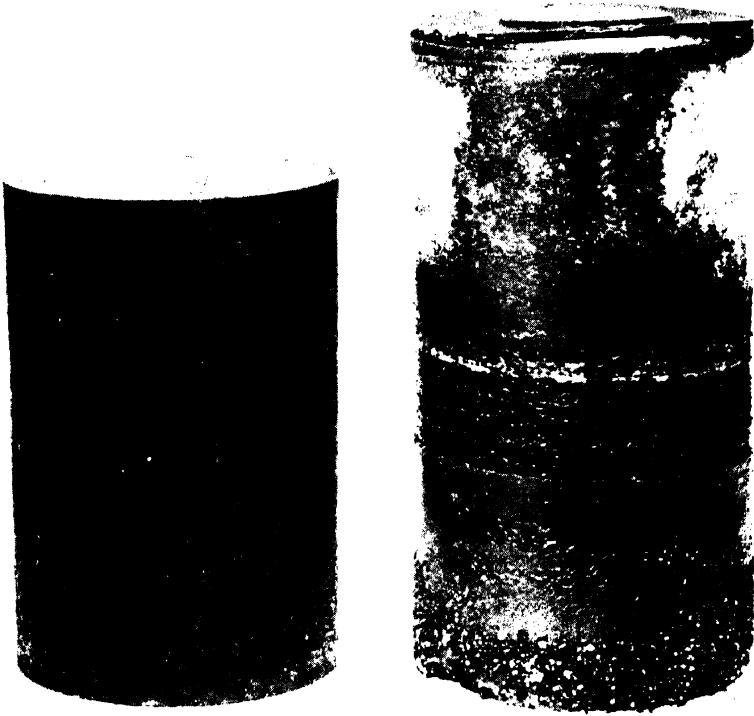
18·8 In the climatic conditions in this country, the penetration of frost under roads seldom exceeds 12 to 14 in. A thickness of construction of this order is usually required on main roads to provide adequate stability. For this reason cases of frost damage to such roads are comparatively rare. Owing to the normally high strength of chalk, a thickness of construction of 8 in. or less is commonly used for heavily trafficked roads built on this subgrade. Consequently, numerous cases of frost damage to main and secondary roads on chalk occur during severe winters. Examples of such damage are shown in Chapter 7. It is clear that if main roads are to be designed in the future to obviate the risk of frost damage, a minimum thickness of 12 to 14 in. of frost-resistant material will be necessary.

18·9 Frost heave on existing secondary roads presents a difficult problem. Cost often precludes reconstruction to a safe thickness, and occasional trouble from frost has to be tolerated. In general, if traffic is diverted or restricted during the thaw, the damage can be minimized. It is usual now for local authorities to keep records of roads liable to frost damage in their areas. Such records, extending over a number of severe winters, disclose any areas which are particularly troublesome and which it may be desirable to improve.

18·10 It does not appear that the slightly different thermal properties of bituminous and concrete roads affect materially the depth of frost penetration under the climatic conditions of the British Isles. The uniform thickness of concrete roads results in uniform heave where the subgrade is homogeneous. Where soil conditions change over the width or length of a slab tipping of the slab may occur, support being provided at isolated points only. Differential heaving of the surface is common on many old macadam roads owing to variations in the thickness of construction.

DAMAGE CAUSED BY THE ENTRY OF WATER THROUGH A SURFACE RENDERED PERMEABLE BY THE ACTION OF FROST

18·11 Successive freezing and thawing may cause certain types of flexible surfacings to craze and become porous. This particularly applies to certain medium- and open-textured carpets which are already somewhat porous. When this occurs, water entering the cracks may affect the stability of either the construction material or the subgrade. The cracks then open under traffic loads, and a complete failure of the road may follow unless the traffic is diverted immediately and the surface is re-sealed. A typical failure of this type is shown in Plate 18·3.

18·12 This form of damage is particularly likely to arise where water from melting snow, which has been allowed to remain on the surface during the frost period, is unable to escape after the thaw owing to snow-clogged drains. In the absence of snow, the danger of serious trouble arising appears to be small except where the thickness of construction is barely adequate for the type of subgrade and traffic loads to be carried.

(a) Before freezing

(b) After 10 days' freezing

FROST HEAVE IN BRICKEARTH

Initial height of specimen 6 in. Heave 1·2 in.

PLATE 18·1

FROST BOILS
along the centre of a secondary road

PLATE 18·2

EARLY STAGE IN SURFACE DETERIORATION
following the entry of water through surface crazed by frost action

PLATE 18·3

SUMMARY

18·13 Frost damage to road foundations may arise either as a result of frost heave in the soil or as a consequence of water entering the construction through a surface rendered porous (or more porous) by low temperature conditions.

18·14 In certain soils, the suction associated with the freezing process causes water to migrate into the upper soil strata and form horizontal layers or lenses of segregated ice which cause an upward displacement or heave of the surface. Frost heave does not occur in coarse-grained soils or in medium and heavy clays. Chalk, and silty sands with particle-size curves falling within limits given in this chapter are likely to be most affected under the climatic conditions of the British Isles.

18·15 Damage resulting from the entry of water through a surface crazed by low temperature conditions is most likely to occur where melting snow is allowed to lie on the road, and where the thickness of construction is barely adequate for the subgrade and traffic conditions.

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CHAPTER 19

THE MEASUREMENT OF SOIL STRENGTH

GENERAL OUTLINE

Introduction

19·1 The behaviour of soil under stress is generally more complex than that of other materials with which the civil engineer has to deal. Not only do soils of different types differ considerably in their resistance to deformation under stress, but such deformation depends upon the moisture content, bulk density and internal structure of the soil and upon the way in which the stress is applied. Furthermore, the soil beneath a foundation is seldom homogeneous and large variations in strength may occur in both the vertical and horizontal planes.

19·2 The variation of the shear strength, as the stress normal to the shear plane is increased, is the usual basis of measurement.

The Relationship between Shear Stress and Normal Stress in Soils

The Mohr Circle

19·3 Consider a small element of a material subjected to minor and major principal stresses σ_x and σ_y (Fig. 19·1(a)). By resolving the forces acting on the triangular wedge ABC, the normal stress σ_n and the shear stress τ, acting on the plane BC, at an angle α with the direction of stress σ_x, are obtained as:—

$$\left.\begin{aligned}\sigma_n &= \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_y - \sigma_x}{2}\cos 2\alpha \\ \tau &= \frac{\sigma_y - \sigma_x}{2}\sin 2\alpha\end{aligned}\right\}$$

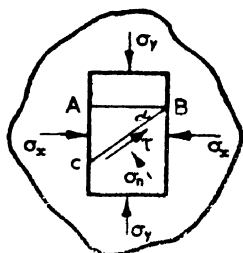



FIG. 19·1 (a) STRESSES ON SMALL ELEMENT OF MATERIAL

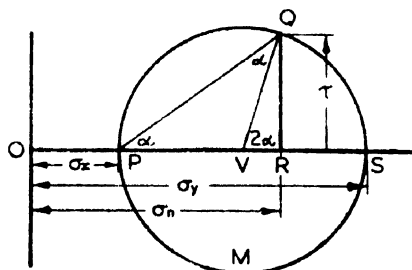



FIG. 19·1 (b) MOHR CIRCLE CONSTRUCTION FOR THE STRESSES ON PLANE BC (FIG. 19·1(a))

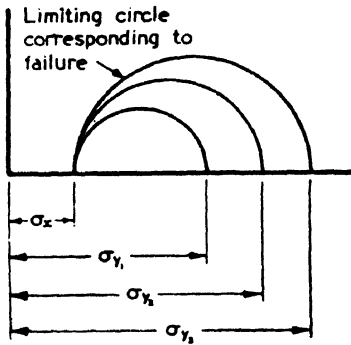



FIG. 19·2(a) MOHR CIRCLES FOR FIXED σ_x AND VARIOUS VALUES OF σ_y (FIG. 19·1(a))

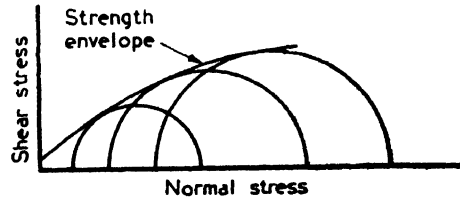



FIG. 19·2 (b) STRENGTH ENVELOPE OF LIMITING MOHR CIRCLES

These stresses can be represented graphically in terms of the principal stresses σ_x and σ_y and the angle of inclination of the plane, by the Mohr circle construction (Fig. 19·1(b)). OP and OS represent the stresses σ_x and σ_y respectively and PQ, at an angle α with OS, cuts the circle on PS at Q. Then the length of QR, the vertical from Q on to OS, represents the shear stress τ, on the plane BC, and OR the normal stress σ_n. This follows since

$$OR = OV + VR$$

$$= \sigma_y - \frac{\sigma_y - \sigma_x}{2} + \frac{\sigma_y - \sigma_x}{2} \cos 2\alpha$$

$$= \frac{\sigma_y + \sigma_x}{2} + \frac{\sigma_y - \sigma_x}{2} \cos 2\alpha$$

and

$$QR = \frac{\sigma_y - \sigma_x}{2} \sin 2\alpha$$

As the plane BC (Fig. 19·1(a)) is rotated, the normal and shear stresses on the plane are defined by the point Q as it moves on the circle PQSM (Fig. 19·1(b)).

19·4 For every combination of the principal stresses σ_x and σ_y, the stress condition on the element under consideration will be represented by a different Mohr circle. Thus if σ_x remains constant while various values of σ_y are applied the stress conditions will be represented by a series of circles (Fig. 19·2(a)). Under these circumstances there will be a limiting value of σ_y at which failure will occur and this stress condition will give rise to the limiting circle (Fig. 19·2(a)). This circle, whilst including the combination of normal and shear stresses which has given rise to failure, does not indicate the inclination of the plane of failure. If failure is obtained with various combinations of the principal stresses σ_x and σ_y, each failure condition will be represented by a limiting Mohr circle and the envelope of all these circles will give the relationship

between the shear stress and normal stress in the element at failure (Fig. 19·2(b)). This envelope is referred to as the strength envelope, or envelope of rupture.

19·5 In the discussion above, the stresses in a small element only have been considered. If, however, the material can be considered to be homogeneous and the principal stresses uniformly distributed, the Mohr circle construction can be used to obtain the shear and normal stresses on a finite plane.

Coulomb's Equation for Soils

19·6 For soils, the strength envelope approximates to a straight line (Fig. 19·3), which can be represented by the equation:—

$$s = c + \sigma_n \tan \phi$$

where s = shear stress at failure (generally referred to as the shear strength)

σ_n = the normal stress

c = the intercept on the axis representing shear stress

ϕ = the slope of the straight line with respect to the axis representing normal stress.

This equation is usually referred to as Coulomb's equation. In the past c has been described as the cohesion and ϕ as the angle of internal friction of the soil,

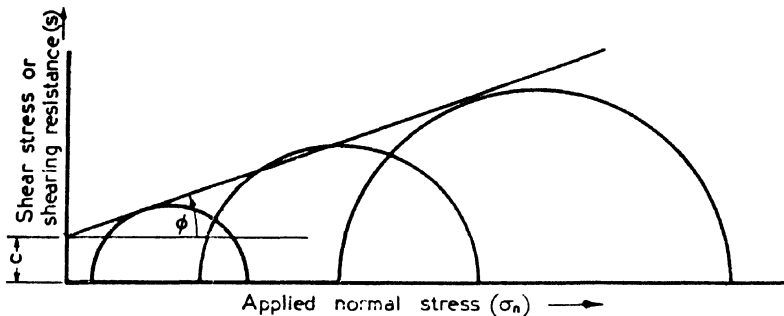



FIG. 19·3 STRENGTH ENVELOPE FOR SOIL

and the misconception has arisen that these are fundamental properties of the soil. It is important to realize that this is not the case, and that the c and ϕ of Coulomb's equation are merely constants derived from the geometry of the graph obtained by plotting shear stress against normal stress. The values of c and ϕ for any given soil initially at a given moisture content and bulk density depend on the conditions under which the stress is applied to the soil. For these reasons it is now generally agreed that c should be termed the apparent cohesion and ϕ the angle of shearing resistance

Modified form of Coulomb's Equation

19·7 In the above form of Coulomb's equation it is assumed that the normal stress acting across the shear plane creates the increase in the shear strength of the soil, whereas, in fact, it is due to the effective stress acting across the plane. The effective stress σ_e is the total component normal to the plane of the forces

transmitted through the points of contact of the grains within unit area. In a saturated soil the normal stress is not necessarily equal to the effective stress, since the application of stress will usually result in a temporary increase in the pore-water pressure with a corresponding reduction in the effective pressure. The pore-water pressure u is usually measured in terms of the pressure in the pore-water in excess of atmospheric pressure. When a pore-water pressure exists in a stressed soil the following relationship holds:—

$$\sigma_e = \sigma_n - u$$

A modified and more general form of Coulomb's equation is therefore:—

$$s = c + \sigma_e \tan \phi$$

If σ_n is replaced by σ_e, the equation is represented graphically by Fig. 19·3.

True Cohesion and Angle of Internal Friction of Soil

19·8 When an element of soil is subjected to a constant minor principal (effective) stress σ_x, and an increasing major principal (effective) stress σ_y the stress conditions at various stages during the process can be represented by a series of Mohr circles as shown in Fig. 19·4, in which the limiting Mohr circle PQR represents the condition of failure. If the element fails on a plane inclined at angle α to the direction of the minor principal stress, it has been shown above that the shear stress, s, and the normal stress σ_e acting on this plane are given by the co-ordinates of the point Q when PQ is drawn at angle α to the horizontal axis in Fig. 19·4.

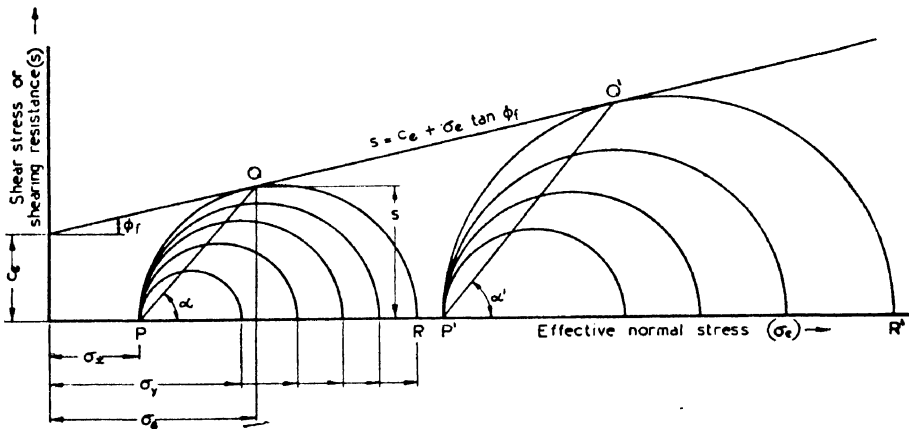



FIG. 19·4 CRITERION OF FAILURE FOR SOIL

19·9 The generally accepted criterion of failure for an element of soil subjected to compound stresses is that failure occurs on the plane on which the following equation is first satisfied:—

$$s = c_e + \sigma_e \tan \phi_f$$

where c_e is defined as the true cohesion
and ϕ_f is defined as the true angle of internal friction,

both being constants for the soil at the condition of bulk density and moisture content occurring at failure. This equation can be plotted as a straight line (Fig. 19·4) and, since the co-ordinates of the point Q give the stresses acting on a failure plane, the straight line must pass through the point Q. Furthermore, this line must be tangential at the point Q to the circle PQR (otherwise it would be possible to draw a smaller circle which would cut the line and so represent failure conditions) and from the geometry of the figure

$$\alpha = 45^\circ + \frac{\phi_t}{2}$$

19·10 Similarly it is possible to draw another set of Mohr circles corresponding to a greater but constant minor principal stress. If it could be arranged that the condition of the element of soil at failure was identical to that in the first case considered, then the limiting circle P′Q′R′ (Fig. 19·4) would also be tangential to the line $s = c_e + \sigma_e \tan \phi_t$ at the point Q′ and the inclination of the failure plane, α′, would also equal $45^\circ + \frac{\phi_t}{2}$. Not all authorities on soil mechanics would accept this definition of the true cohesion and true angle of internal friction; but it is considered that values obtained according to this procedure would approximate closely to fundamental values of soil strength.

19·11 These values are not easily determined in practice, and in any case they would have little practical application, since it is not possible to determine accurately the soil conditions which are likely to exist at failure. It is therefore best to recognize that the shear properties determined by laboratory tests are not fundamental but depend on the test conditions, which should therefore be selected to suit the field conditions being considered.

Typical Strength Envelopes

19·12 Typical strength envelopes for three different soils are shown in Fig. 19·5. All the specimens for each soil had the same initial moisture content and bulk density.

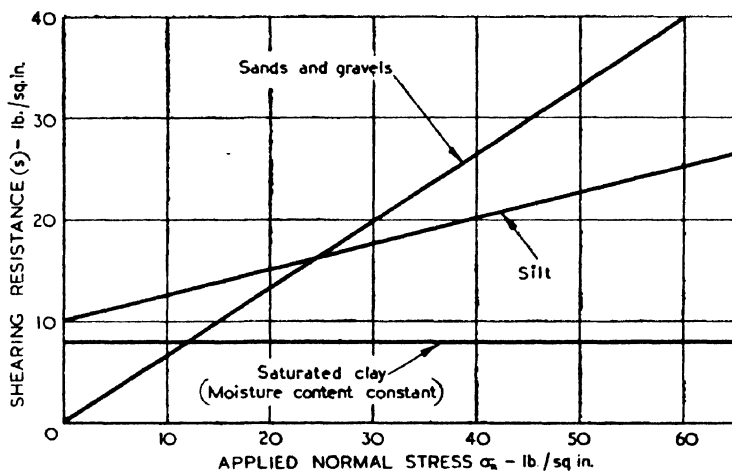



FIG. 19·5 TYPICAL SHEARING RESISTANCE/NORMAL STRESS RELATIONSHIPS

19·13 CLAYS. It has been well established that saturated clays, when tested under conditions allowing no changes in moisture content, exhibit a constant shearing resistance, i.e. they behave as though purely cohesive, their angle of shearing resistance being zero (Fig. 19·5). On the other hand, the inclination, α, of the plane of failure to the direction of the minor principal stress in such tests is seldom 45° and may be as much as 58°. This shows that the true angle of internal friction, ϕ_t, of clay is seldom zero and may be as much as 26° $\left(\alpha = 45° + \frac{\phi_t}{2}\right)$[1].

19·14 Saturated clays tested under conditions in which complete drainage occurs during the test have angles of shearing resistance up to 30°.[2] Under these conditions the effective stresses equal the applied stresses but it does not follow that the measured angle of shearing resistance equals the true angle of internal friction. The specimens subjected to the higher normal stresses will have lower moisture contents and higher bulk densities at failure than those subjected to lower normal stresses so that the true cohesion and angle of internal friction may change from specimen to specimen, this change being responsible for part of the increase in shearing resistance with increasing normal stress.

19·15 Little is known about the fundamental properties of unsaturated clays but they can be shown experimentally to have positive angles of shearing resistance even when tested under conditions allowing no change in moisture content.

19·16 SANDS AND GRAVELS. Dry or saturated clean sands and gravels have zero true and apparent cohesion and so all envelopes pass through the origin (Fig. 19·5). The angle of internal friction for dense sands varies between 35° and 46° and for loose sands between 28° and 34°[2]. That for dense well graded gravel may be as much as 50°[3].

19·17 The angle between the horizontal and the slope of a heap produced by pouring dry sand from a small height, known as the angle of repose, is approximately equal to the angle of internal friction of the sand in a loose state. This can be proved as follows: suppose the angle of repose of such a sand is β (Fig. 19·6). An element of the sand ABC of weight w outside the surface would shear on the plane AC, the normal and shear stresses at failure being given by

$$\sigma_n = \frac{w \cos \beta}{AC}$$

$$\text{and } s = \frac{w \sin \beta}{AC}$$

Since the material has zero cohesion,

$$s = \sigma_n \tan \phi_t$$

$$\therefore \frac{w \sin \beta}{AC} = \frac{w \cos \beta}{AC} \tan \phi_t$$

$$\therefore \beta = \phi_t$$

Damp sands usually have a small apparent cohesion which may be explained as the frictional resistance brought into play by surface tension forces.

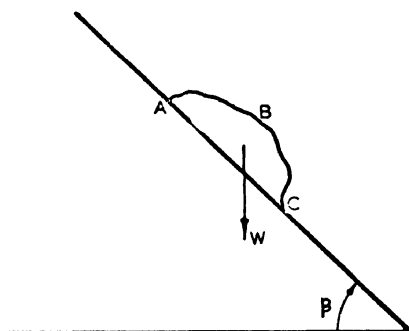



FIG. 19·6 ANGLE OF REPOSE OF COHESIONLESS SAND

19·18 SILTS. Soils such as silts which can be considered as intermediate between clay and sand have positive values of cohesion and angle of shearing resistance, even when tested under conditions allowing no change in moisture content (Fig. 19·5). Their angles of shearing resistance and of internal friction usually lie between those common for sands and those common for clays.

The Stress/Strain Relationship for Soils

19·19 Detailed consideration has not so far been given to the deformation which occurs when a soil is stressed, or to what condition of deformation constitutes "failure." The stress/strain relationship for soil is of particular interest to the road engineer who is concerned primarily with repeated applications of stresses considerably smaller than those required to cause failure.

Single Application of Stress

19·20 If a cylindrical specimen of soil is subjected to a constant lateral stress, σ_x, and an increasing longitudinal compressive stress, σ_y, a curve of the type shown in Fig. 19·7 may be obtained when the longitudinal strain is plotted against the longitudinal stress. The fall to an ultimate stress after the peak stress has been reached only occurs with some soils and can only be recorded with some types of test apparatus. The peak stress is usually taken as the failure stress or strength of the soil. The first part of the curve often approximates to a straight line and, by analogy with the stress/strain relationship for metals, the stress corresponding to the limit of the straight part of the curve is termed the limit of proportionality. Although over this range the strain is proportional to the longitudinal stress, the total strain is not recoverable as it usually is with metals.

19·21 The stress/strain relationship for different soils is shown in Fig. 19·8. For sands (curve I) it is generally not possible to distinguish an initial straight portion to the curve. The peak stress at failure is, however, well defined. For heavy saturated or nearly saturated clays (curve II) the peak stress remains sensibly constant for a considerable increase in strain so that the soil fails

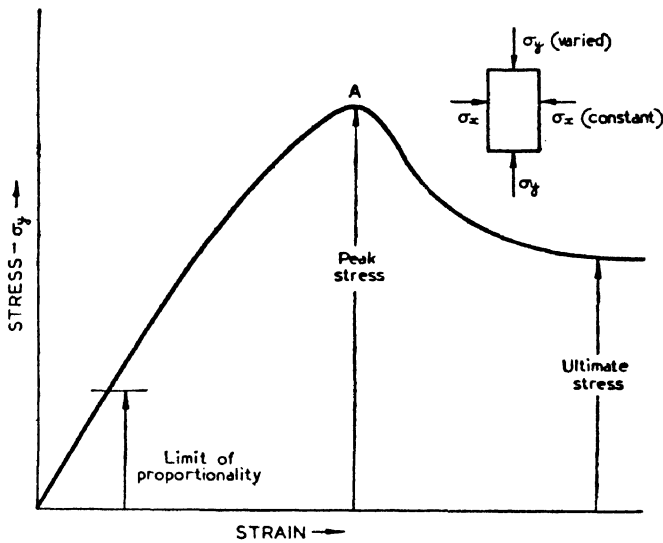



FIG. 19·7 STRESS/STRAIN RELATIONSHIP FOR SOIL

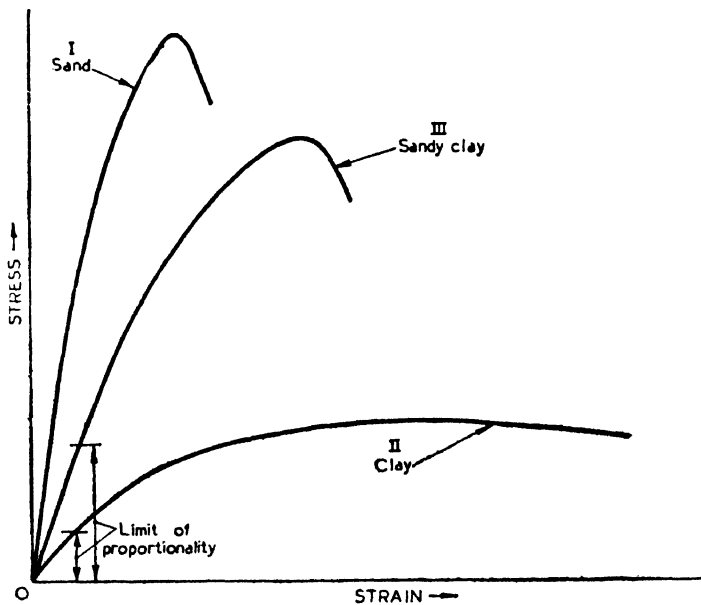



FIG. 19·8 TYPICAL STRESS/STRAIN RELATIONSHIPS FOR A SINGLE APPLICATION OF COMPRESSIVE LOAD TO VARIOUS SOILS

plastically. An initial straight portion to the curve can sometimes be distinguished. Soils intermediate between sands and clays and dry unsaturated clays produce a relationship (curve III) in which a fairly well defined peak stress is reached and in which it is often possible to distinguish an initial straight portion to the curve.

19·22 MODULUS OF DEFORMATION. The slope of the initial straight portion of the stress/strain curve is generally referred to as the modulus of deformation, since most of the strain is non-recoverable. Where the range of stress covered by the straight portion is insufficient, or where there is no discernable straight portion at all, an approximate modulus can be taken as the slope of the straight line which most nearly fits the curve over the range of stress considered.

19·23 PERMANENT DEFORMATION. A permanent deformation usually remains after a stress less than the ultimate has been applied and released. The strain is only completely recoverable for very small stresses, showing that soils have only limited elastic properties. The permanent deformation is the result either of an increase in the bulk density of the soil or of plastic flow.

Repeated Application of Stress

19·24 If a stress less than the peak stress is applied to a specimen of soil and released a number of times, the stress only remaining on for a limited period, there is a gradual alteration in the shape of the stress/strain curve. Fig. 19·9 shows how the modulus of deformation and the limit of proportionality increase and the additional permanent strain decreases for repeated application of the same stress to a specimen of clay. If, after a certain number of load cycles, the additional permanent strain with each load cycle remains constant, this must be due to plastic flow. The greater additional permanent strains occurring during earlier load cycles are partly due to an increase in the bulk density, which must involve a closer packing of the soil particles and therefore increased rigidity of the soil structure. This explains the gradual increases in the modulus of deformation and the limit of proportionality, and the gradual decrease in the additional permanent strain during these earlier load cycles. If the applied stress is sufficiently small, the hysteresis loop will close after a number of load cycles, i.e. there will be no further increase in the permanent strain and, provided the stress/strain curve is then a straight line, the specimen will behave elastically for that range of stress.

19·25 I should be noted that the general shape of a diagram such as that in Fig. 19·9, and the number of load cycles required to reach equilibrium conditions, depend upon the rate of application and duration of the applied stress. Similar series of stress/strain curves can be obtained for sands, except that for such soils there will seldom be any straight portion to the curve, although this may not prevent the closing of the hysteresis loop after a certain number of load cycles, provided the applied stress is sufficiently small. If repeated loads are applied to the surface of a mass of soil, the graph of deformation of the surface against load has a similar shape to that described for stress and strain of a specimen.

Types of Strength Test

19·26 The tests used to determine the strength properties of soil can be divided into three broad groups: shear tests, bearing tests and penetration tests.

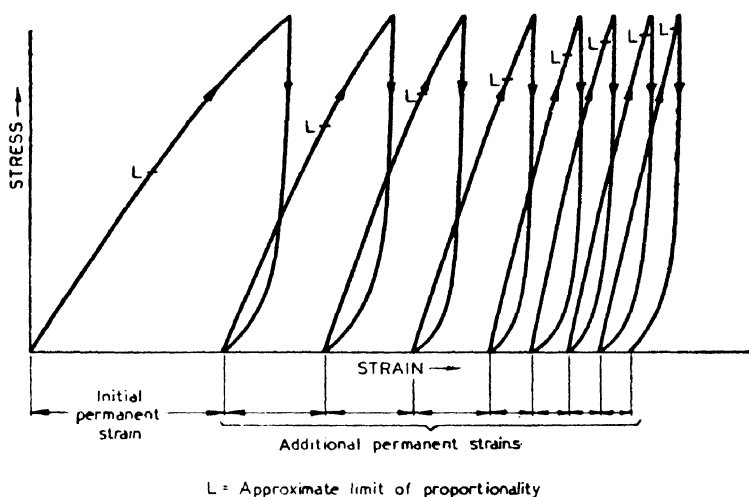



FIG. 19·9 TYPICAL STRESS/STRAIN RELATIONSHIP FOR REPEATED APPLICATION OF COMPRESSIVE LOAD TO SANDY CLAY

Shear Tests

19·27 These tests are usually made on comparatively small samples in the laboratory and thus only determine effectively the strength properties at a point in the soil mass. A number of tests on samples from different points must be made in order to obtain an overall evaluation of the strength of the soil mass. The usual object of shear tests is to determine the apparent cohesion and angle of shearing resistance under test loading and drainage conditions similar to those that will occur in the soil mass. These constants are then used in conjunction with some theory of stress distribution or theory of plastic failure in the soil mass. Some shear tests can also be used to determine the modulus of deformation for use in theories of stress distribution in an elastic medium requiring a value of the modulus of elasticity.

Bearing Tests

19·28 These are loading tests made in the field on the surface of the soil mass. The results are therefore affected by variations in soil properties within the volume of soil stressed, and give an overall measurement of the strength of this part of the soil mass. The results of bearing tests are not necessarily used in conjunction with a theory of stress distribution or ultimate failure, since in an ideal bearing test the intensity of loading and size of loaded area are equal to those of the structure under consideration.

Penetration Tests

19·29 These tests are made either in the field or in the laboratory and, like shear tests, only determine effectively the strength properties at a point in the soil mass. Penetration tests may be considered as small-scale bearing tests in

which the ratio of penetration to size of loaded area is much greater than in bearing tests.

19·30 The results of penetration tests must usually be correlated either with results of shear tests to obtain values of c and ϕ, or with past experience of the behaviour of structures on soil of similar strength, as in the case of empirical methods of designing the thickness of road or airfield pavements.

Factors Affecting the Results of Soil Strength Tests

Factors associated with the Actual Tests

19·31 The factors affecting the results of soil strength tests which are associated primarily with the tests themselves are:—

(1) Size and shape of the specimen.

(2) Method of loading.

(3) Rate of loading.

(4) Drainage conditions.

Items (1) and (4) are only applicable to laboratory tests.

19·32 Factors of secondary importance which usually have little effect are climatic conditions such as temperature, humidity and atmospheric pressure.

19·33 SIZE AND SHAPE OF SPECIMEN. The effect of the size of the specimen depends upon the type of soil being tested. So long as the dimensions of the specimen are much greater than those of the coarsest particles in the soil, it is assumed that the size of specimen has little effect on the test results. However, the smaller the specimen the greater is the chance that it will not be truly representative since soil samples, especially undisturbed samples, are seldom uniform in properties. The shape of the specimen differs from test to test. The best shape gives a uniform distribution of stress and reduces edge effects to a minimum.

19·34 METHOD OF LOADING. Two methods of loading are employed in strength tests on soils: controlled-strain and controlled-stress. In the controlled-strain method, the strain to be varied is applied continuously, usually by some form of screw jack acting through a proving ring to measure the stress. In the controlled-stress method, the stress is usually applied in increments by dead loading. It is only possible to determine the ultimate and the peak value of the stress by using the former method.

19·35 RATE OF LOADING. At rapid rates of loading, the shearing resistance and modulus of deformation of cohesive soils increase with the rate of loading[4]. This effect can be attributed to viscosity of the soil. It is much less marked with sands. At slower rates of loading, the strength of saturated soil may increase with decreasing rate of loading but this is related to the drainage conditions during the test.

19·36 DRAINAGE CONDITIONS. The greater the degree of drainage attained during a test on a saturated soil, the greater will be the shearing resistance because the drainage is accompanied by consolidation which increases the bulk density and reduces the moisture content causing an increase in both the cohesion and the angle of internal friction.

Factors associated with the soil

19·37 The factors affecting the test results which are associated with the soil rather than the test are:—

(1) Soil type.
(2) Dry density of the soil.
(3) Moisture content of the soil.
(4 Permeability of the soil.
(5) Structure formed by the soil particles.

19·38 SOIL TYPE. The strength of soils of different types has a general tendency to decrease with decreasing particle size.

19·39 DRY DENSITY AND MOISTURE CONTENT OF THE SOIL. The strength of any one soil usually increases with increasing dry density and decreasing moisture content; the effects of dry density and moisture content are, however, to a certain extent inter-related.

19·40 PERMEABILITY OF THE SOIL. The permeability of the soil determines the rate at which drainage occurs and its effect on the results of strength tests is therefore covered by the remarks on drainage conditions given above.

19·41 STRUCTURE FORMED BY THE SOIL PARTICLES. The strength of a soil containing clay remoulded at the natural moisture content and compacted to the natural dry density may be considerably less than the strength of the undisturbed soil. This loss in strength is due to destruction during remoulding of the natural structure formed by the soil particles. (The unconfined compressive strength of an undisturbed clay divided by the strength of the remoulded clay is termed the degree of sensitivity. For most clays this ratio ranges between 2 and 4 but exceptionally is as much as 8[(2)]).

19·42 Natural soil strata are often anisotropic so that the strength of undisturbed specimens may be affected by the direction in which they were taken. This effect can also occur with specimens cut from larger samples of remoulded and compacted soil, when the compactive effort has been applied in one direction only.

SHEAR TESTS

Introduction

19·43 The object of shear tests is generally to determine the values of the apparent cohesion and angle of shearing resistance of soil under known test conditions. Two methods are commonly used:—

(1) SHEAR BOX TEST. In the shear box failure is caused in a predetermined plane of the soil, the shear strength or shearing resistance and the normal stress both being measured directly.

(2) TRIAXIAL TEST. In this test longitudinal and lateral stresses are applied to a sample of the soil, and these stresses determine the plane of failure. The strength envelope is obtained by using different combinations of the applied stresses.

The Shear Box Test

19·44 The essential feature of the apparatus is a rectangular box, divided horizontally into two halves and containing a rectangular prism of soil. While the prism is subjected to a constant vertical compressive force, an increasing horizontal force is applied to the upper half of the box, thus causing the prism to shear along the dividing plane of the box. A diagram of the apparatus is shown in Fig. 19·10. The test is made on a number of identical specimens using different vertical stresses so that a graph of shearing resistance against vertical stress can be plotted.

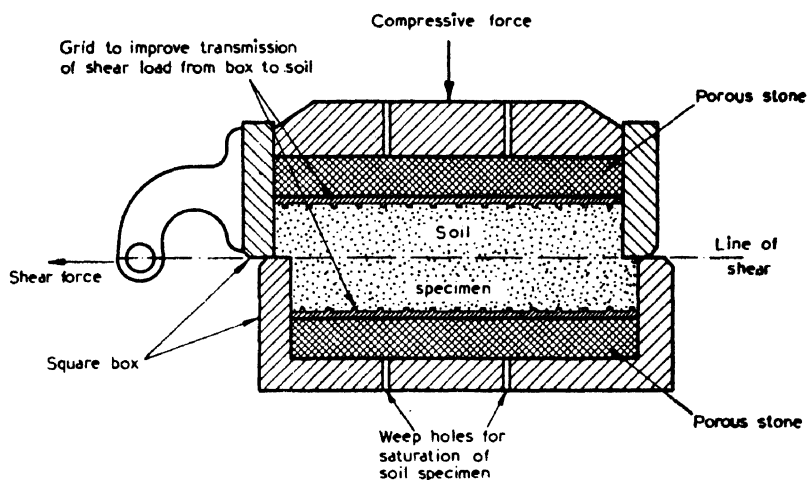



FIG. 19·10 DIAGRAM OF SHEAR BOX

Apparatus

19·45 DESIGN OF BOX. For soils ranging from clays to coarse sands the dimensions of the specimen normally used are 6 cm. x 6 cm. x approx. 2 cm.; for gravels a shear box developed for testing a specimen 12 in. x 12 in. x 6 in. has been found satisfactory[3].

19·46 APPLICATION OF LOADS. The constant compressive stress is obtained by means of weights attached to a stirrup which bears on the plate above the specimen. The mode of applying the shear load depends on whether the test is of the controlled-stress or controlled-strain type. The latter is now the most usual, and the shear load is applied by a screw jack (preferably motor-driven and equipped with interchangeable gear ratios) acting through a proving ring. The transfer of shear load to the specimens is assisted by perforated or plain metal grids.

19·47 MEASUREMENT OF DEFORMATION. Many forms of shear box apparatus are equipped with dial gauges to measure both the shear deformation and the vertical contraction or expansion of the specimens.

19·48 APPARATUS USED AT THE ROAD RESEARCH LABORATORY. Of the two forms of apparatus used at the Road Research Laboratory, that shown in Plate 19·1A applies the shear load by means of a lever and jockey weight system

and is of the controlled-stress type, and that shown in Plate 19·1B applies the shear load cont nuously by means of a hand or motor-driven screw jack and is of the controlled-strain type. In the latter apparatus, the shear box is contained in a trough so that the soil specimen may remain in contact with free water during the test, if so required. This apparatus is made to a City and Guilds College design.

Preparation of Specimens

19·49 The problems of specimen preparation are common to all soil strength tests. Consequently, many of the statements made here with reference to the shear box test apply also to the other soil strength tests. When the strength of cohesive soil in its natural state is required, every effort should be made to obtain undisturbed specimens, but if these are unobtainable, remoulded specimens must be used and an approximate correction, based on experience with similar soils, made for the loss in strength due to remoulding. When the soil is to be disturbed and compacted, as for an embankment, the strength should be determined from remoulded specimens. With sands and gravels there is usually no objection to remoulding, provided the natural dry density and moisture content are known.

19·50 UNDISTURBED SPECIMENS. Undisturbed specimens may be obtained by carefully trimming larger masses of soil down to the required size of prism. Alternatively, they may be obtained by pressing the upper half of the box, fitted with a special square cutter, into the ground.

19·51 REMOULDED SPECIMENS. Remoulded specimens may also be obtained by trimming down a larger mass of soil which has been compacted in a large mould to the required dry density at the required moisture content. Alternatively, soil at the required moisture content may be compacted to the required dry density in the box itself. Because of the difficulty of obtaining a uniform dry density and the tendency for planes of weakness to form parallel to the shear plane, this second method is not very satisfactory for cohesive soils.

Test procedure

19·52 The aim in selecting the test procedure for use with the shear box apparatus is to make the conditions of rate of loading, drainage, etc. in the test, reproduce those in the field as closely as possible. However, exact reproduction is not obtainable because, on the one hand, the conditions in the field cannot always be fully predicted or are not fully known, and on the other hand, the range of techniques at present available with the shear box test (and other shear tests) is limited.

19·53 In practice, shear box tests are divided into two groups, undrained and completely drained, according to the drainage permitted during shearing of the specimen. These are also known as immediate or quick, and slow tests respectively. If the specimens in an undrained test are consolidated in the boxes prior to shearing, the test is known as a consolidated-undrained or consolidated-quick test.

19·54 IMMEDIATE, UNDRAINED OR QUICK TESTS. In this type of test the horizontal shear loading is begun immediately after the vertical compressive load has been applied. For saturated soils, the aim is to make the rate of shear

loading sufficiently fast for no appreciable consolidation to take place but not so fast as to impart additional shearing resistance due to viscosity. These two aims cannot be completely reconciled. A suitable rate of deformation for clays is usually in the region of 0·02 in./min. When the rate of loading in practice is very rapid, as it is with road or airfield pavements, it may be advisable to make the rate of loading in the shear tests approximate to that under the actual structure and thus include some viscous resistance in the measured shearing resistance.

19·55 In this type of test, consolidation is also restricted by the use of solid metal grids next to the specimen, thus helping to prevent the escape of pore water. As there should be no consolidation, it is not necessary to measure vertical movements during the test and even the measurement of horizontal movement is not always essential.

19·56 True immediate tests in the shear box apparatus cannot be made on highly permeable soils such as sands because the specimen consolidates before shearing is complete even with the use of the solid metal end-plates. Such tests must be made in the triaxial apparatus. In true immediate tests on saturated soils which are sufficiently impermeable for no drainage to take place, it follows that the volume of the specimen remains unaltered. However, in immediate tests on unsaturated soils, some compaction will almost certainly take place during the test, so that in this case the volume of the specimen decreases.

19·57 CONSOLIDATED-QUICK OR CONSOLIDATED-UNDRAINED TESTS. This type of test differs from the immediate or quick type in that the specimen is allowed to consolidate under the vertical load before any shear load is applied. The specimens of the series are then sheared as in the immediate test, at a rate sufficient to prevent any further change in moisture content. Again, these tests are not possible on permeable soils in the shear box apparatus and must be made in the triaxial apparatus. In this type of test it is necessary to measure the vertical movement during the consolidation in order to ascertain when the latter is complete. Drainage of the specimen during consolidation is facilitated by the use of perforated grids or porous plates next to the specimen.

19·58 SLOW OR DRAINED TESTS. In this type of test a saturated specimen is first allowed to consolidate under the vertical load and then sheared sufficiently slowly for any further changes in moisture content to take place, i.e. for the pore pressure to remain zero. These tests can be made very easily on sand specimens which consolidate almost immediately and with which a comparatively rapid rate of shearing can be used. With clays, using the 6 cm. x 6 cm. shear box of the motor-driven controlled-strain type, the test takes 1 to 2 days for the initial consolidation and 6 to 8 hours for shearing (data from City and Guilds College).

Results

19·59 The results of a series of tests on a number of identical specimens, using the same test procedure except for varying the normal load, are plotted as shearing resistance against normal stress and a straight line is drawn through the points. The equation of this line is $s = c + \sigma_n \tan \phi$ where c is the

apparent cohesion and ϕ the angle of shearing resistance. In reporting results for c and ϕ the type of test and other test details should always be given because the values for any soil vary with the test conditions, as previously discussed.

19·60 It should be noted that only in the case of slow or drained tests is the applied normal stress, σ_n, equal to the effective normal stress, σ_e. In immediate or undrained tests on saturated clays, the change in the pore-water pressure should equal the applied normal stress so that the effective stress is zero. Under these circumstances the angle of shearing resistance is zero, and the shearing resistance, s, is equal to the apparent cohesion, c (Fig. 19·5). It also follows that, in the case of consolidated-undrained tests on saturated clays, it is the value of the normal stress applied during consolidation that determines the shear strength of a particular specimen, and that increasing the normal stress for the shearing part of the test will have no effect on the shear strength, since it only produces a corresponding increase in the pore-water pressure.

19·61 In the case of immediate tests on unsaturated soils, $\sigma_e \leq \sigma_n$ so that $\phi > 0$, but it should be remembered that some compaction is almost certain to occur during shearing. A typical result for an unsaturated sandy clay is given in Fig. 19·11.

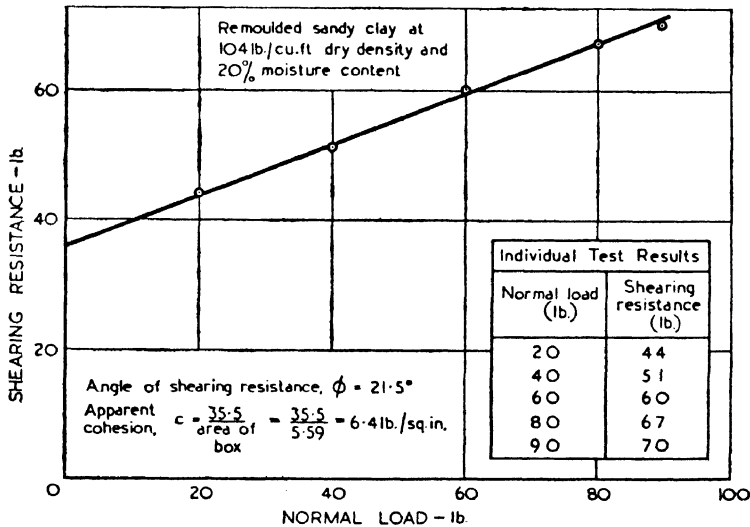



FIG. 19·11 TYPICAL SHEAR BOX RESULTS FOR IMMEDIATE TESTS

The Triaxial Compression Test

19·62 In the triaxial compression test, a specimen of soil is subjected to three compressive stresses at right angles to one another, and one of these stresses is increased until the specimen fails in shear. The test differs from the shear box test in that the plane of shear failure is not predetermined.

19·63 In the usual types of triaxial apparatus, cylindrical specimens are subjected to constant radial stresses generated by fluid pressure and an increasing axial stress generated by some loading system. As with the shear box test, a number of identical specimens must be tested; in the triaxial test, each specimen usually has a different constant radial stress applied to it. The axial stress is increased until each specimen fails and the stress conditions at failure are analysed by Mohr circles as discussed in para. 19·4, so that the apparent cohesion and angle of shearing resistance can be determined. Again, as with the shear box, the test conditions are selected to correspond as closely as possible with the field conditions and similar procedures to those already mentioned for the shear box are used.

Apparatus

19·64 A typical pressure cell is shown in Fig. 19·12. Plates 19·2 and 19·3 show the controlled-strain type of apparatus used at the Road Research Laboratory and Fig. 19·13 a controlled-stress type.

19·65 Shape and size of specimen. All designs use a cylindrical specimen. The length:diameter ratio of the cylinder usually lies between 2 and 2·5; the normal length of 3 in. is suitable for most types of soil, but specimens up to 8 in. long are sometimes used for tests on coarse-grained soils.

19·66 Design of pressure cell. Since the radial stress is applied to the specimen by fluid pressure, the specimen is contained in a pressure cell and sealed from the fluid by a sleeve, preferably made of pure rubber. To ensure that negligible additional constraint is placed on the specimen this sleeve should be as thin as is consistent with complete impermeability and resistance to damage by sharp coarse soil particles. A suitable thickness for specimens of 1½-in. diameter is 0·015 in. The walls of the cell are usually made of a transparent material such as perspex so that the specimen can be kept under observation throughout the test. Water is normally the fluid used to provide the radial stress, but compressed air or glycerine is sometimes used.

19·67 Measurement of stress and deformation. The radial stress or fluid pressure is measured by a pressure gauge or a manometer. The axial stress is measured by a proving ring or similar device if the load is applied at a steady rate through a screw jack (controlled-strain) and by weights if the load is applied in increments as dead loading (controlled-stress). In most types of apparatus the fluid pressure acts on the top of the specimen as well as on the sides, so that the total axial stress on the specimen is the stress applied externally, measured by the proving ring or other device, plus an additional stress due to the fluid pressure.

19·68 In some forms of apparatus, the change in the pore-water pressure inside a saturated specimen can be measured during the test. The specimen is capped top and bottom with porous plates and perforations in the seatings are connected to a mercury manometer, the connection being filled with water. The far side of the manometer is connected to a pressure gauge or a second manometer and to a supply of compressed air. The two levels in the first manometer are maintained equal by regulating this supply of compressed air. Hence, no pore-water is allowed to flow out of the specimen although its pressure is recorded on the pressure gauge. Alternatively, the porous plates and outlets

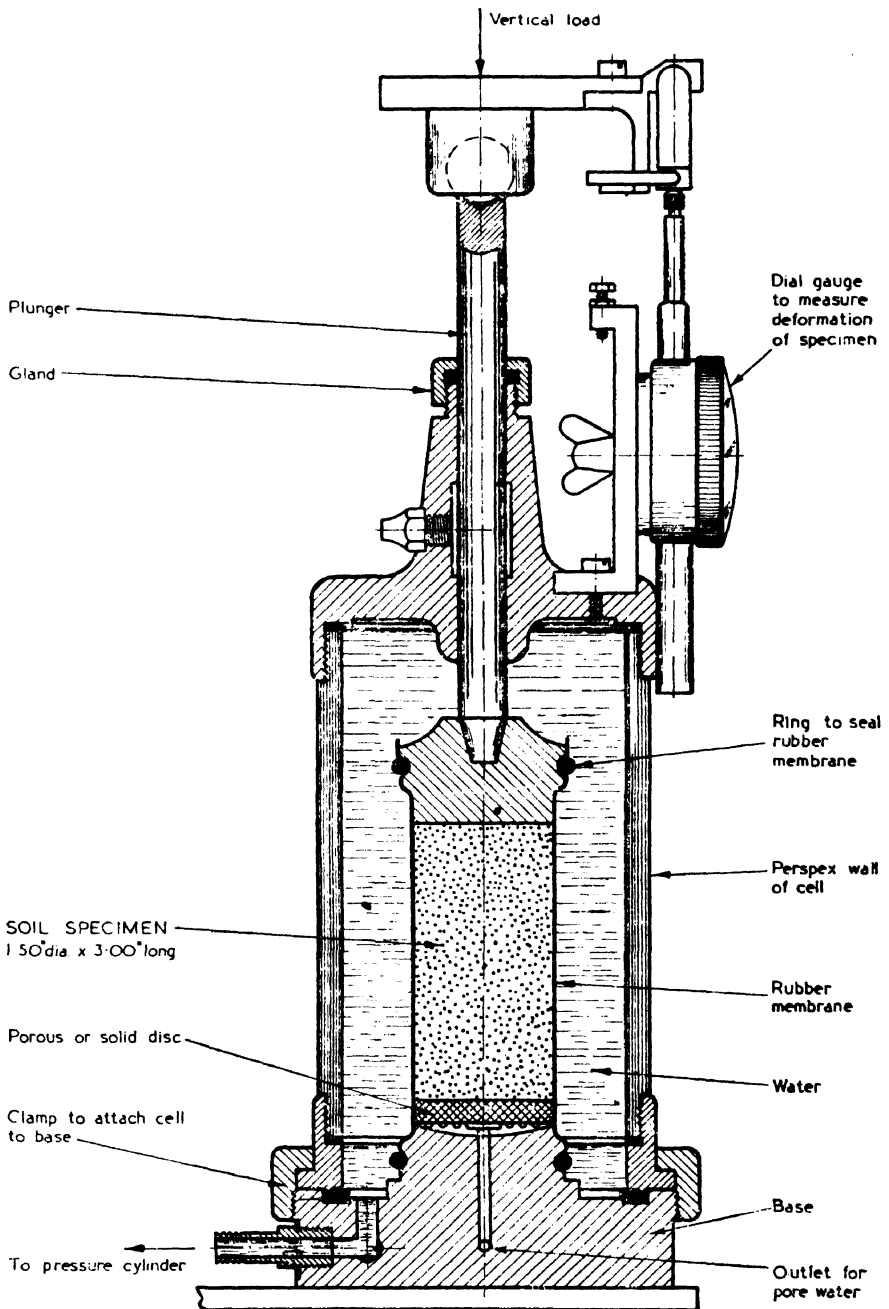



FIG. 19·12 TYPICAL TRIAXIAL COMPRESSION CELL

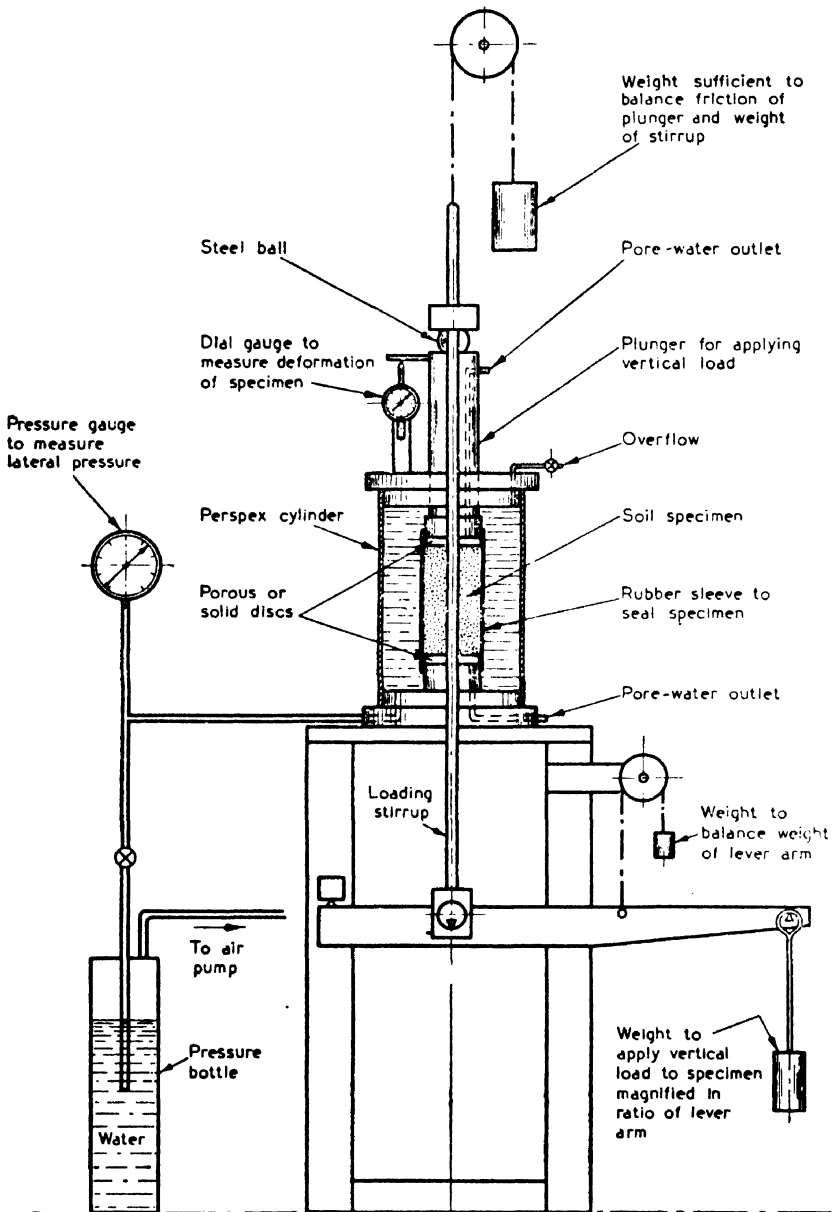



FIG. 19·13 TRIAXIAL COMPRESSION MACHINE, CONTROLLED-STRESS TYPE

from the perforations in the seatings may be connected to the lower end of a burette, the upper end of which is open to the atmosphere. The volume of pore-water expelled and hence the change in volume of a saturated specimen during a test can thus be measured.

19·69 The axial deformation is usually measured by a dial gauge acting on a bracket attached to the plunger used to apply the axial load.

Specimen Preparation

19·70 The remarks already made on the uses of undisturbed and remoulded specimens for the shear box test also apply to the triaxial test.

19·71 UNDISTURBED SPECIMENS. Undisturbed specimens are obtained by means of hollow cylindrical cutters. In order to minimize the disturbance of the soil, the wall thickness of the cutters is made as small as possible. Specimens may be obtained directly in the field, the cutters being sealed top and bottom for transit to the laboratory, or a larger undisturbed sample of soil may be taken in a mould of, say, 6-in. diameter and the cutters used in the laboratory.

19·72 The cutters used at the Road Research Laboratory are of two types. One type has an internal diameter of either 2·00 or 1·50 in. throughout, while the other type has this diameter only over the first ¾-in. length, beyond which the internal diameter is relieved to either 2·02 or 1·52 in. This serves to reduce the adhesion of the soil to the walls of the cutter and so facilitates the extrusion of the specimen. Both types have external diameters of either 2·15 or 1·62 in. The relieved type of cutter is most suitable for highly cohesive soils whereas the unrelieved type is most suitable for soils of low cohesion which require the full support of the cutter walls and in any case are easier to extrude.

19·73 REMOULDED SPECIMENS. Remoulded specimens are obtained in one of two ways: either by compacting soil to the correct shape and density in a constant-volume mould, or by pressing the hollow cylindrical cutters into soil remoulded in a larger mould. (For methods of remoulding soil in moulds of 6-in. diameter, see details of specimen preparation for the California bearing ratio test, p.384.) The first method is usually the easier but there is less tendency for the density to vary throughout the specimen when it is prepared by the second method.

Test Procedure

19·74 IMMEDIATE, UNDRAINED OR QUICK TESTS. In this type of test the specimen, sealed in a rubber sleeve, is capped top and bottom with impervious plates so that no pore-water can escape, and vertical loading is started immediately after a constant lateral pressure has been applied. As with the shear box test, part of the resistance of the specimen to deformation may be due to viscosity. The rate of loading must therefore be sufficiently slow for this resistance to be negligible unless it is desired to include some viscous resistance in the measured strength. Readings of deformation and vertical load are taken at frequent intervals until the specimen fails. The test is repeated on 3 to 5 identical specimens at different constant lateral pressures.

19·75 In the triaxial apparatus, consolidation can be effectively prevented, even in sands, so that true immediate tests can be made on all soils. However,

immediate tests on unsaturated soils will not necessarily be under constant-volume conditions because the air in the specimen may be compressed. A more elaborate form of immediate test can be made in which measurements of pore pressure are also taken, using the apparatus already described. This is comparatively easy with saturated sands but the technique is more difficult with saturated clays and virtually impossible with all unsaturated soils. It should be noted that the pore pressure is sometimes negative because dense sands and silts tend to dilate when sheared. The measurement of pore pressure enables the effective stresses to be calculated in addition to the applied stresses. A simpler procedure for immediate tests on saturated sands, which gives the effective stresses within the specimen, is to connect the pore-water outlet from the specimen to a water-filled burette open to the atmosphere, as described for measuring volume changes in saturated specimens, and to keep the level in the burette constant by varying the lateral pressure during the application of the vertical load[5]. Thus the volume of the specimen remains constant, i.e. no drainage takes place, but also the pore pressure remains zero and therefore the applied vertical stress and lateral pressure are effective stresses throughout the test.

19·76 CONSOLIDATED-QUICK OR CONSOLIDATED-UNDRAINED TESTS. In this type of test, a saturated specimen is first free to drain through the pore-water outlet and is consolidated under a constant lateral pressure. When no further pore-water is expelled, i.e. when consolidation is complete, the outlet for the pore-water is closed and the specimen is subjected to increasing axial load under constant moisture content or undrained conditions as in the quick or immediate test. The consolidated-quick test in the triaxial apparatus is thus similar to the same test in the shear box except that consolidation is in three directions instead of only one. (This may have a significant effect on the results.) As with the immediate triaxial test, this test can be made on all soils and it is also possible to perform a more elaborate test involving the measurement of the pressure of the pore-water during axial loading.

19·77 SLOW OR DRAINED TESTS. In this type of test, a saturated specimen is first allowed to consolidate under constant lateral pressure only. The axial load is then applied sufficiently slowly for all further consolidation to take place before the specimen fails. The change in volume of the specimen is measured from the volume of pore-water expelled. Slow tests on clays in the triaxial apparatus take longer than similar tests in the shear box—usually a week for initial consolidation and several days for axial loading.

Test Results

19·78 CORRECTION FOR VARYING CROSS-SECTIONAL AREA. For immediate tests on saturated specimens it can be assumed that the volume of the specimen remains constant and the average cross-sectional area at any deformation can be calculated. In the case of immediate tests on unsaturated specimens this assumption is not quite correct but the cross-sectional area corrected on this assumption at least gives a closer approximation to the true value of the axial stress than the uncorrected original area. Calculations for the corrected area are similar to those for an unconfined compression test specimen given in the appendix. This correction and the calibration of the proving ring are used to construct a special chart on which lines of constant proving ring reading are

FIG. 19·14 TYPICAL RESULTS OF TRIAXIAL TEST ON ROAD RESEARCH LABORATORY MACHINE

drawn on a graph of stress against deformation. Such a chart with a series of typical results plotted for quick tests is given in Fig. 19·14. The total axial stress on the specimen is the sum of the stress read from the data sheet and the lateral pressure. This chart is due to A. W. Bishop of City and Guilds College.

The average cross-sectional area during axial loadings in consolidated-quick tests can be calculated, assuming the volume of the specimen remains constant, from the average area after initial consolidation is complete. The average cross-sectional area during slow tests must be calculated from the measured volume changes.

19·79 MODULUS OF DEFORMATION. The modulus of deformation of the soil can be determined by dividing the slope of the initial linear part of the stress/deformation curve by the original length. For an approximate value of the modulus of deformation for a specimen having little or no linear part to its stress/deformation curve, the best straight line should be drawn through all the points over the required range of stress and the modulus calculated from the slope of this line.

19·80 TYPES OF FAILURE. The nature of the failure in a triaxial specimen is related to the shape of the stress/deformation curve. A brittle failure (curve I —Fig. 19·8) takes place at a relatively small deformation on a single cone or plane, as in Plate 19·4A. A plastic failure takes place at a much greater deformation on an infinite number of parallel planes giving a barrel shape as in Plate 19·4B. An intermediate case in which the specimen has become barrel-shaped but has actually failed on a finite number of planes is shown in Plate 19·4C.

FIG. 19·15 DEDUCTION OF APPARENT COHESION AND ANGLE OF SHEARING RESISTANCE FROM THE RESULTS OF A SERIES OF TRIAXIAL TESTS

(a) Controlled-stress type

(b) Controlled-strain type

SHEAR BOX APPARATUS

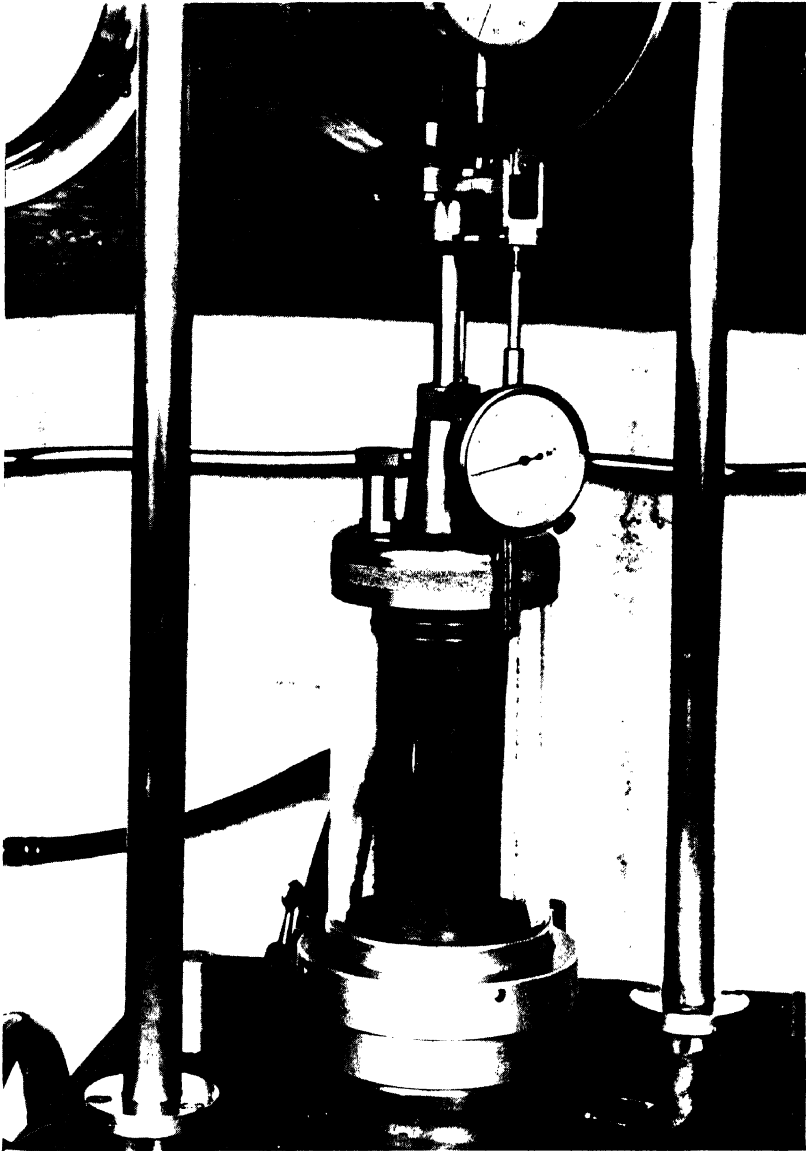
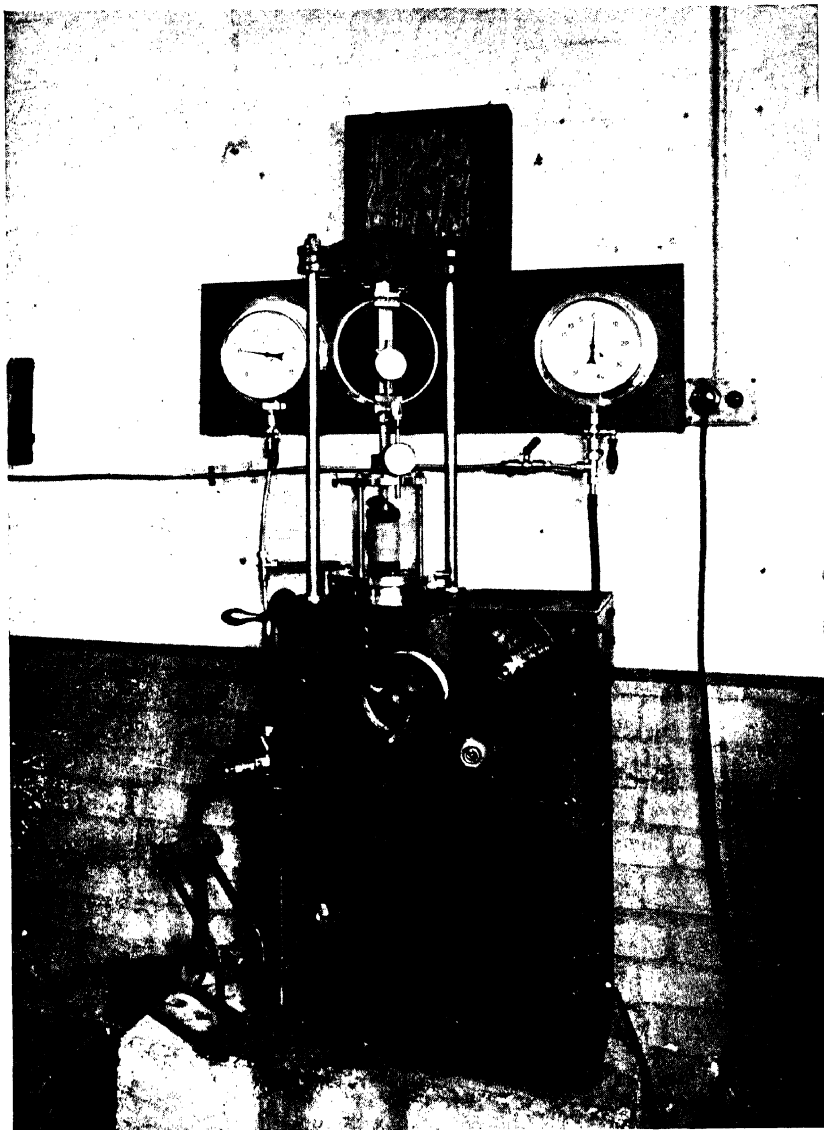
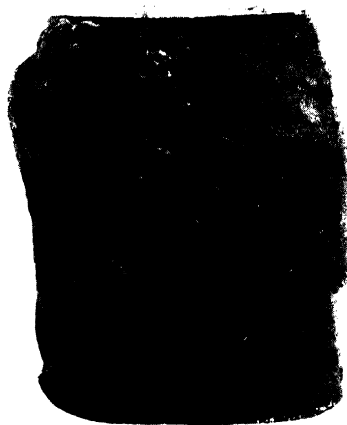
PLATE 19·1

ROAD RESEARCH LABORATORY TRIAXIAL COMPRESSION MACHINE OF CONTROLLED-STRAIN TYPE
(close-up view showing cell for 2-in. diameter soil specimens)

PLATE 19·2

ROAD RESEARCH LABORATORY TRIAXIAL COMPRESSION MACHINE OF CONTROLLED-STRAIN TYPE

(General view showing cell for $1\frac{1}{2}$-in. diameter soil specimens)

PLATE 19·3

(a) Brittle failure

(b) Plastic failure

(c) Intermediate type of failure

TYPICAL TRIAXIAL SPECIMENS AFTER FAILURE

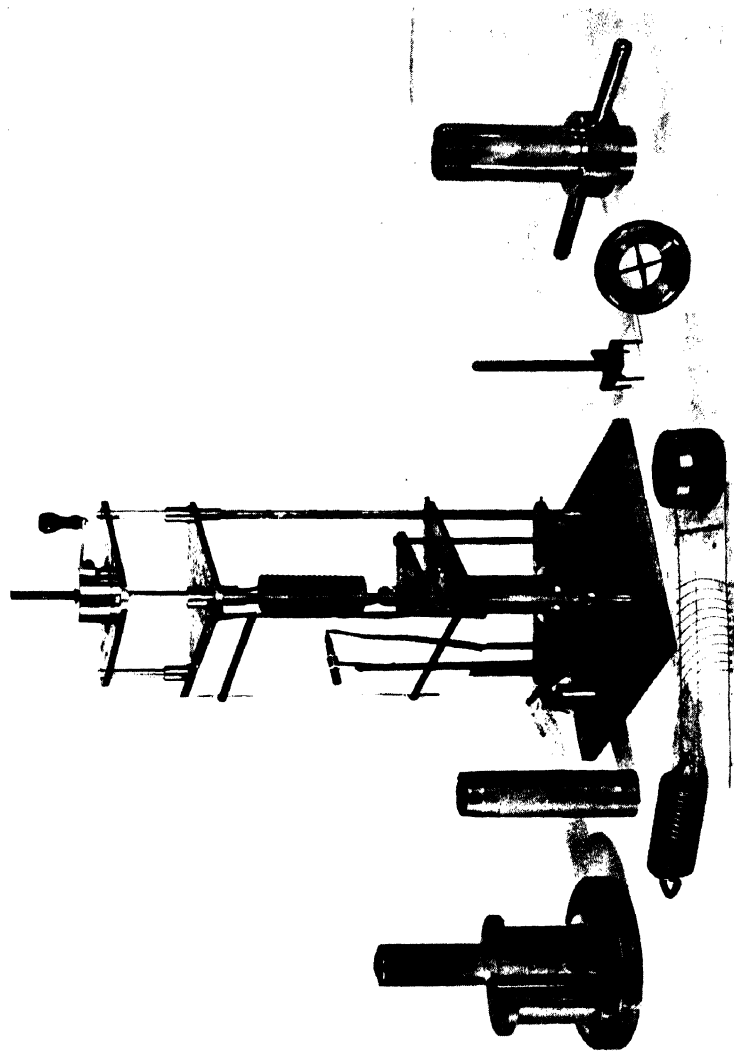



PLATE 19·4

UNCONFINED COMPRESSION APPARATUS

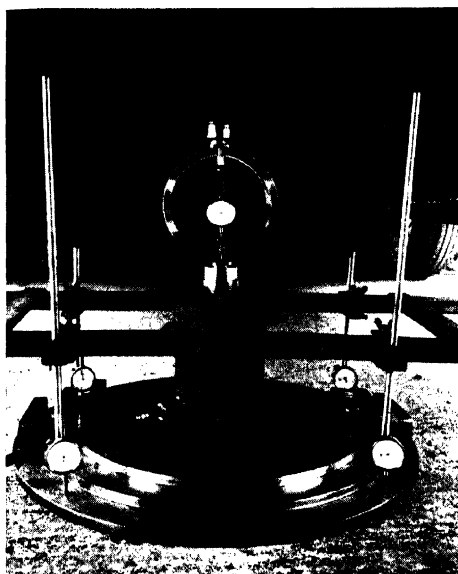



PLATE 19·5

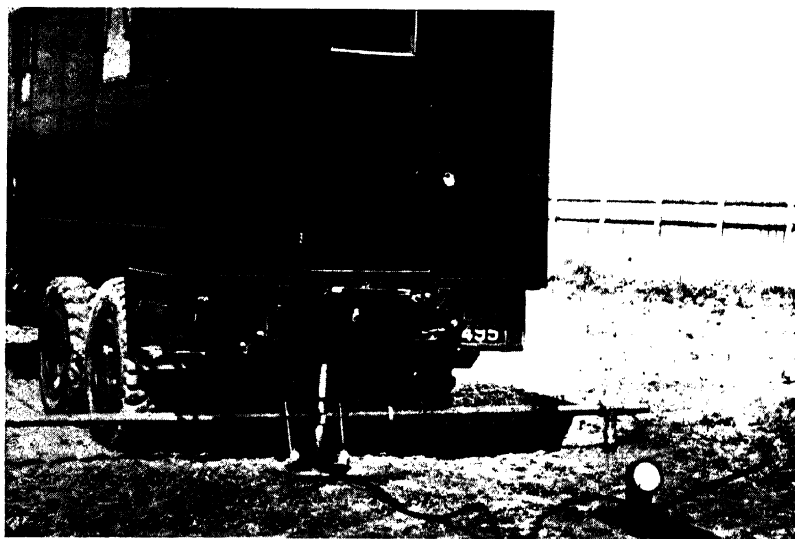
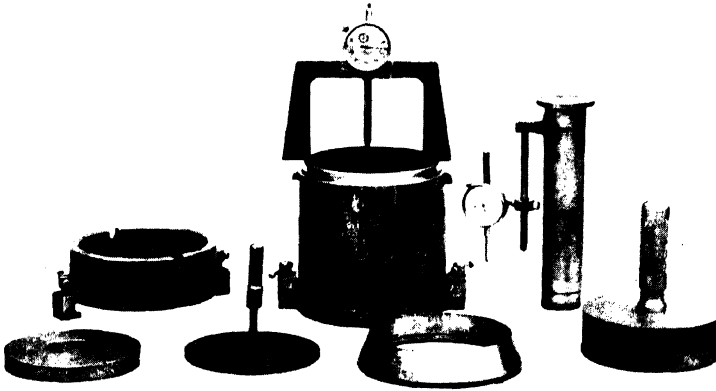
(a) Test with plate of 30-in. diameter

(b) Test with plate of 12-in. diameter

PLATE BEARING TEST APPARATUS

PLATE 19·6

(A) C.B.R. MOULD AND TEST EQUIPMENT

(B) PENETRATION TEST ON SPECIMEN IN MOULD USING COMPRESSION TESTING MACHINE

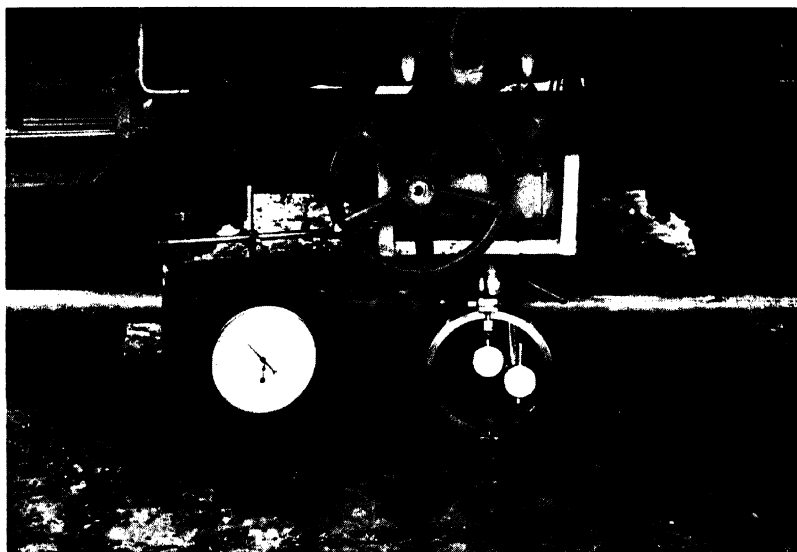
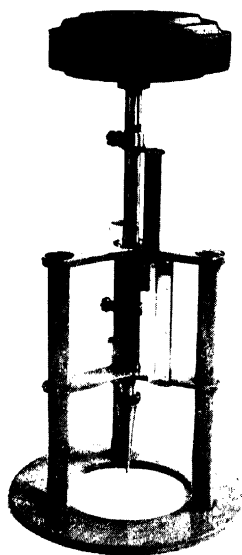
PLATE 19·7

(A) IN SITU C.B.R. APPARATUS

(B) NORTH DAKOTA CONE APPARATUS

PLATE 19·8

19·81 APPARENT COHESION AND ANGLE OF SHEARING RESISTANCE. The construction of Mohr circles is described in para. 19·3. To obtain the apparent cohesion and angle of shearing resistance from a set of triaxial results, the Mohr circle for the stress condition at failure of each specimen is plotted and the best common tangent to these circles is drawn. The apparent cohesion is then the intercept of this line on the vertical axis and the tangent of the angle of shearing resistance is the slope of this line. The Mohr circles for the results in Fig. 19·14 are plotted in Fig. 19·15. The general remarks regarding the values of the apparent cohesion and angle of shearing resistance determined with the shear box using different types of test also apply to the triaxial test.

19·82 INCLINATION OF PLANE OF FAILURE. The inclination of the plane of failure to the plane on which the radial stress acts equals $45° + \frac{\phi_t}{2}$. In immediate tests on saturated clays, $\phi = 0$ but ϕ_t for such soils may be as much as 26° so that the inclination of the plane of failure is not 45° and may be as much as 58°. This provides an approximate method of determining the true angle of internal friction in immediate tests by measuring the actual inclination of the plane of failure in the specimen[1].

The Unconfined Compression Test

19·83 The unconfined compression test is a special case of the triaxial compression test in which the axial compressive stress only is applied to the cylindrical specimen. Since the apparatus requires no provision for applying lateral pressure, and since the specimen need not be contained in a rubber membrane, the unconfined compression test has been developed as a simple field test. The apparatus is useful only for immediate tests on predominantly clayey soils which are saturated or nearly saturated.

Apparatus

19·84 A diagram of a typical unconfined compression machine is given in Fig. 19·16. This form of apparatus, developed at the Building Research Station[6], is in common use in this country. It is portable, and quick and simple to use. A photograph of the machine together with its accessories—spare spring, cutters for obtaining undisturbed specimens, extruding device and coning tool for trimming specimens—is shown in Plate 19·5.

19·85 SPECIMENS. The specimens are cylindrical, 1½ in. in diameter with ends hollowed in the form of cones. The angle of the cones is such that the overall length of the specimen is 3⅝ in. and the length between the apices of the cones is 3 in.

19·86 LOADING. The specimen is loaded between two brass end-plates, the lower plate being movable but the upper one fixed. The end-plates are coned to fit the ends of the specimen. The effect of coning is to reduce the tendency of the specimen to become barrel-shaped by counteracting the effect of friction between the plates and the soil. The load is applied through a calibrated spring by turning the handle of the simple screw jack at the top of the machine. As the screw moves the upper moving plate upwards, the spring draws up the middle and lower moving plates and the specimen is thus compressed between the lower moving plate and the lower fixed plate, the load applied being proportional to the extension of the spring.

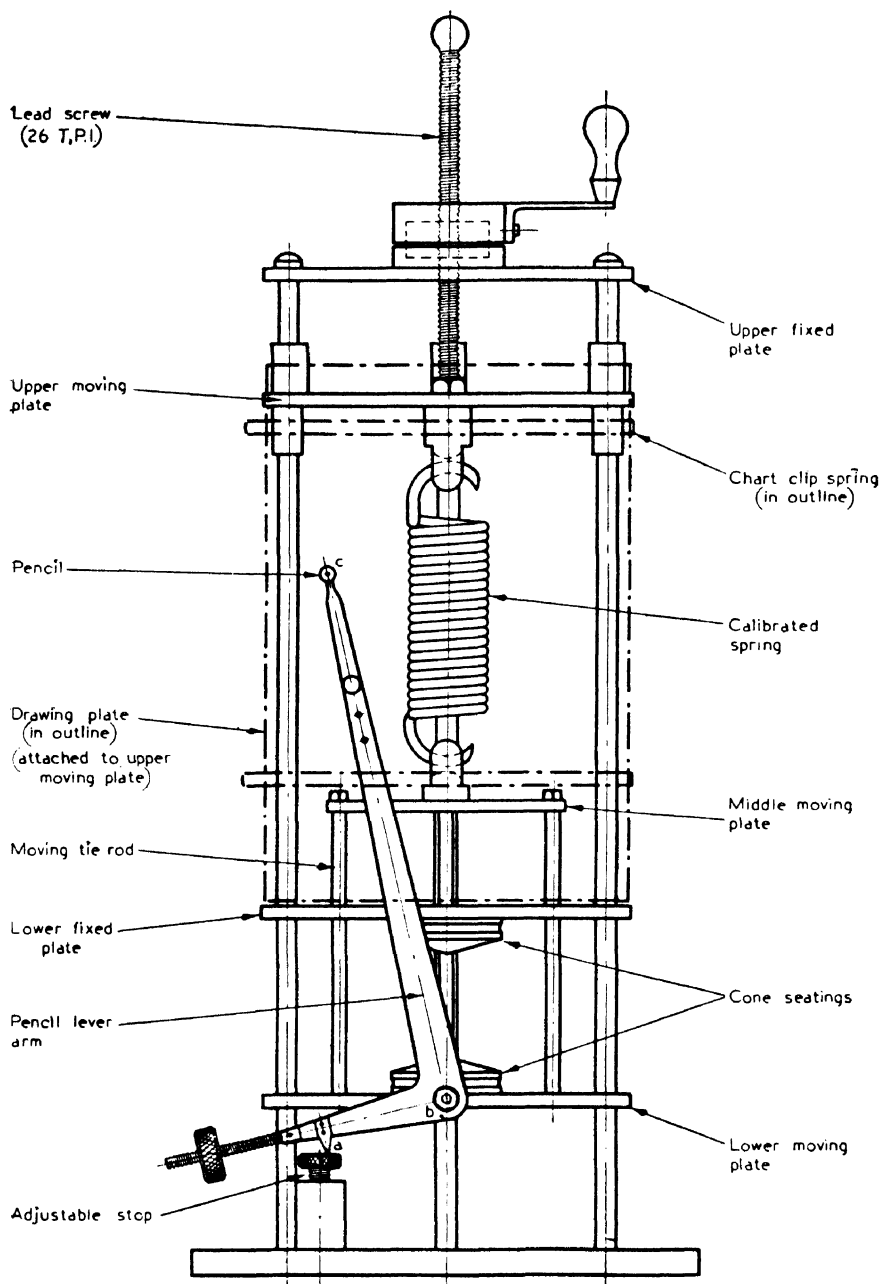



FIG. 19·16 UNCONFINED COMPRESSION APPARATUS

19·87 SPRING. For testing soils of widely different strengths it is convenient to have a number of springs of different stiffness. The choice of spring in any particular case depends upon the experience of the operator. A set of four springs is used at the Road Research Laboratory, of stiffness 10, 25, 50 and 100 lb. load per inch extension. The springs should have a linear load/extension characteristic over approximately 2-in. extension.

19·88 AUTOGRAPHIC RECORDING. The load/deformation characteristic of the specimen is recorded autographically by a pencil moving across a chart on the vertical drawing plate. Referring to Fig. 19·16, the chart holder is fixed to the upper moving plate and the pivot of the pencil arm is fixed to the lower moving plate. The movement of the pencil relative to the chart or drawing plate in a vertical direction is thus equal to the extension of the spring and is proportional to the load applied.

19·89 The deformation of the specimen is equal to the relative movement between the lower fixed and moving plates and is therefore equal to the movement of the pivot of the pencil arm (point b) relative to the base of the machine. Since the fulcrum (point a) remains at a fixed height above the base, the horizontal movement of the pencil measures the deformation of the specimen magnified in the ratio bc to ab. If the extension of the spring does not increase, i.e. if the load on the specimen does not increase, the pencil will move along an arc whose centre is the pivot (point b) and thus the axis of deformation is this arc, although the axis of load is a vertical straight line.

19·90 CHARTS. It is convenient to have the axes of deformation and load printed on the chart. A set of equally spaced vertical lines form a scale of deformation, and one of a set of equally spaced arcs of correct radius and centre, also printed on the chart, is selected as the axis of deformation. The chart is trimmed to such a shape that when its left-hand side is in contact with the slots for the clips which hold it in place, it is correctly located on the drawing plate. Fig. 19·17 shows the type of chart used at the Road Research Laboratory.

Spec men Preparation

19·91 Since the unconfined compression apparatus is intended for field use, the test is usually made on undisturbed specimens. These specimens are obtained in hollow cylindrical cutters of 1½-in. diameter, identical with those described for obtaining undisturbed samples for the triaxial compression test. Specimens can be taken both from the surface of the subgrade and several feet below it by means of auger holes. The extruder and coning tool shown in Plate 19·5 greatly facilitate the removal of the specimen from the cutter and ensure that the correct length and coning of specimen is obtained.

Test Procedure

19·92 The cones of the machine are lightly oiled before the test and the specimen is placed centrally on the lower cone. The handle is then turned until the upper cone is just in contact with the specimen. By adjusting the stop on which the fulcrum (point a, Fig. 19·16) rests, the pencil is set to the thick right-hand vertical line on the chart (Fig. 19·17). It is also convenient to move the chart itself vertically up or down until the pencil is resting on the intersection of this vertical line and one of the arcs, this arc and vertical line

becoming the axes of deformation and load respectively. The specimen is loaded by turning the handle at a rate of half a revolution per second until failure occurs. As with the triaxial compression test, the failure may be brittle or plastic. When a brittle failure occurs, a definite maximum load is reached after which the specimen fails rapidly on planes inclined at an angle depending on the true angle of internal friction. When a plastic failure occurs, no maximum load is reached (although owing to the increase of cross-sectional area, a maximum stress may be reached) and the test is continued until 20-per cent strain has been obtained, by which time the specimen will have become noticeably barrel-shaped. The load/deformation traces on the chart for these two types of failure are shown in Fig. 19·17.

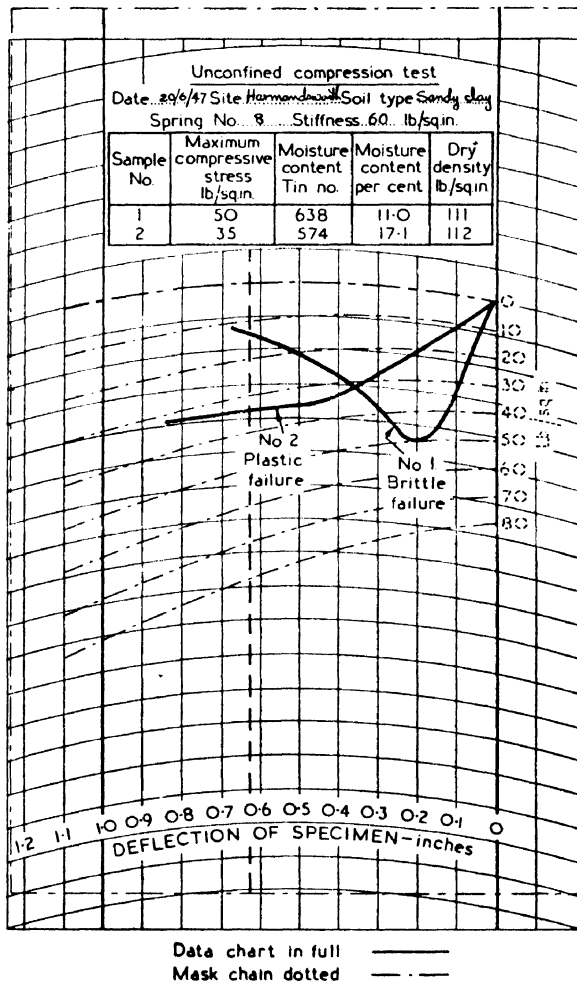



FIG. 19·17 MASK SUPERIMPOSED ON CHART OF TYPICAL RESULTS OF UNCONFINED COMPRESSION TESTS

Results

19·93 Calculation of stress. To calculate the stress in the specimen, it is necessary to allow for the stiffness of the spring and the change in cross-sectional area of the specimen during test. By assuming that the volume of the specimen remains constant and that the shape remains a cylinder, it is possible to calculate the cross-sectional area at any deformation as shown in the appendix. A series of lines of constant stress can then be drawn on transparent material to form a mask so that, when this mask is superimposed on the chart with the axes coincidental, readings of stress can be made. The maximum compressive stress is noted. In the case of a plastic failure in which no maximum stress is reached at less than a strain of 20 per cent, the stress at this strain is noted instead. A typical mask is shown in chain-dotted lines superimposed on the chart in Fig. 19·17.

19·94 Determination of apparent cohesion. The Mohr circle of rupture for an unconfined compression test passes through the origin. In the general case the cohesion, c, and angle of shearing resistance, ϕ, cannot be determined from one circle. However, in the case of immediate tests on saturated clays ϕ is equal to zero, and the envelope of rupture is a straight line parallel to the x-axis at a distance c from it. The circle of rupture then has a radius equal to c. Since the diameter of the circle corresponds to the maximum compressive stress obtained, the apparent cohesion (the shear strength) equals half this stress.

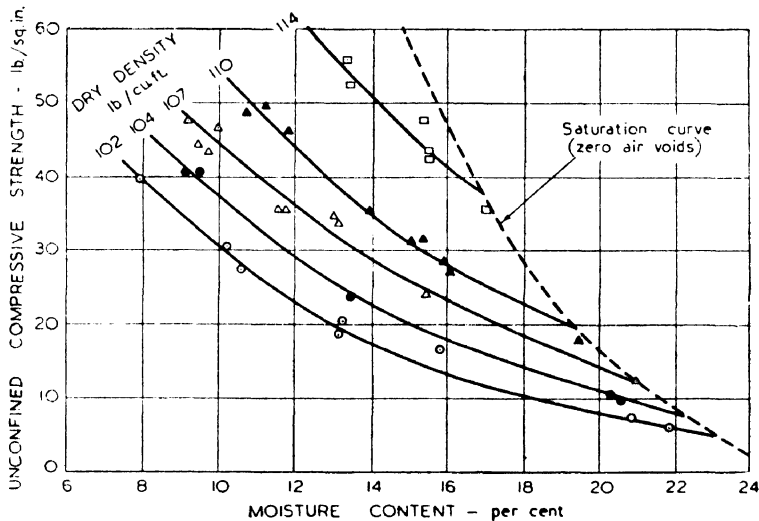



FIG. 19·18 RELATIONSHIP BETWEEN UNCONFINED COMPRESSIVE STRENGTH OF SANDY CLAY AND MOISTURE CONTENT AT DIFFERENT DRY DENSITIES OF THE SOIL

(Laboratory-compacted samples)

19·95 Effect of dry density and moisture content. The results of an investigation made on a sandy clay soil to determine the effect of changes in the dry density and moisture content of the soil on the unconfined compressive strength are given in Fig. 19·18. The specimens were taken by means of cutters

from masses of soil compacted in moulds to known dry densities and moisture contents. It can be seen that within the range of the tests, the unconfined compressive strength increases with dry density but decreases with moisture content. These results are similar to those obtained on most other soil types and with other strength tests. It should be remembered that, even for immediate tests, the specimens far from saturation in this investigation would probably have exhibited appreciable angles of shearing resistance, so that the curves do not represent the full change in immediate strength for all conditions of stress.

Other Forms of Shear Test

19·96 Several other forms of laboratory shear test have been devised amongst which may be mentioned a transverse shear test[7] sometimes used in the U.S.A. as an alternative to the unconfined compression test, and rotary and torsional shear tests[8] occasionally used as alternatives to the shear box and triaxial tests.

19·97 A field shear test for clays which is likely to become of increasing importance is the vane test[9][10][11]. In this test a vane, usually consisting of two blades fixed at right angles, is attached to the end of a rod and pushed into the soil at the bottom of a borehole. The torque required to shear the soil on the surface of the cylinder containing the blades is then measured. The vane test can thus be used to determine the immediate value of the shear strength of clays and is an alternative to the unconfined compression test. However, it has an advantage over the latter test, in that the strength of a clay at considerable depths can be determined without obtaining undisturbed specimens. In fact, although the vane test gives results in good agreement with the unconfined compression test for shallow depths, they are commonly higher than the corresponding unconfined compression test results for great depths. The results obtained with the vane test are regarded as being more reliable because unconfined compression specimens taken from great depths are unlikely to be truly undisturbed.

Application and Selection of Shear Tests

19·98 The principle applications of shear tests may be listed as follows:—

(1) The determination of the ultimate bearing capacity of the soil mass for the design of footings and other foundations (see Chapter 22).

(2) The determination of the stability of embankments, cuttings and other earthworks (see Chapter 26).

(3) The estimation of earth pressures on retaining walls and sub-surface structures.

(4) The design of the thickness of airfield and road pavements. At the present time, however, shear tests other than the unconfined compression test are only used in conjunction with a limited number of infrequently used methods of pavement design (see Chapter 20).

In their application to design problems, shear tests have to be used in some theory of failure or stress distribution within the soil mass[12]. The theories for (1), (2) and (3), and some of the methods for (4) above, usually assume the soil to have a constant apparent cohesion and angle of shearing resistance but, as has already been emphasized, these quantities are not constants but vary

with the test conditions. The selection of the type of test (immediate, consolidated-quick or slow) and the details of test procedure, depend on the site conditions and the nature of the investigation.

19·99 In the present stage of development of the science of soil mechanics, set rules for the application and selection of shear tests cannot be laid down, but the following generalizations can be made.

Immediate Tests

19·100 The results of immediate tests can be applied to problems where critical stresses develop in a saturated soil mass too rapidly for the moisture content to change appreciably, e.g. in the case of the "$\phi = O$ analysis" for saturated clays (see Chapter 26 and references 13 and 14, p. 395).

Consolidated-quick Tests

19·101 The results of consolidated-quick tests can be applied to problems where the soil has consolidated under the load of the structure or under its own weight and critical changes in the stresses within the soil mass then occur too rapidly for any further appreciable changes in moisture content to take place, e.g. rapid draw-down of the water behind a dam.

Slow tests

19·102 The results of slow tests can be applied to problems where the stresses develop within the soil mass sufficiently slowly for all changes in moisture content to take place, e.g. the final bearing capacity of the ground beneath the footings of a building which is erected more slowly than the soil consolidates

Unconfined Compression Test

19·103 The unconfined compression test is generally the most convenient for immediate tests on saturated or nearly saturated clays.

Shear Box Tests

19·104 Immediate or consolidated-quick tests in the shear box when constant-volume conditions are required can be made only on saturated clays; with other soils it is usually impossible to prevent consolidation occurring during shearing. Slow tests can be made on all soils in a reasonably short time. There is a possibility of progressive failure due to non-uniform distribution of stress on the plane of failure. It is not possible to measure the modulus of deformation or elasticity or to measure the pore-water pressure within the specimen.

Triaxial Tests

19·105 All three types of test on all soils can be carried out with the triaxial apparatus. The measurement of the modulus of deformation or elasticity, the measurement of pore-water pressure and the measurement of volume changes are all usually possible. However, the apparatus is elaborate and slow tests take much longer than in the shear box. The triaxial apparatus is probably the most useful for research into the fundamental properties governing the strength of soils since it can be readily adapted to new techniques such as that required for a study of the strength of soil under repeated dynamic loading, as may occur in the subgrades of roads.

BEARING AND PENETRATION TESTS

Introduction

19·106 In both bearing and penetration tests a compressive stress is applied to the soil by a rigid bearing area, the deflection or penetration being measured for various loads. The difference between the two types of test is only a matter of scale. A bearing test is a field test carried out on the natural soil with large-scale apparatus and a test area of the order of several square feet, the soil thus being loaded in much the same manner during the test as in practice. The applied loads cause deflections of the order of 0·1 in. which are partly elastic and partly due to the compression of the soil. Bearing tests associated with road design are seldom continued until large deflections due to plastic failure are produced. A penetration test, on the other hand, is carried out on a small scale in the field or laboratory with a test area of a few square inches. The penetrations caused are of the same order as the deflections produced in bearing tests but since they occur over such a small area they are mainly due to plastic flow. Thus, although each type of test measures a number of fundamental soil properties in unknown proportions, bearing tests measure mainly elastic and unrecoverable compressibility of the soil, while penetration tests measure mainly the resistance of the soil to deformation by shearing.

The Plate-bearing Test to determine Westergaard's Modulus of Subgrade Reaction

19·107 In Westergaard's theory of the stresses and deflections in concrete slabs, the elastic reaction of the subgrade against the slab is assumed to be vertical and proportional at all points to the vertical deflection. The constant of proportionality is called the " modulus of subgrade reaction " (symbol " k ") and is measured in lb./sq.in./in. In actual fact, the modulus of subgrade reaction of a particular subgrade is not a constant so that the test procedure for its determination must be specified. The following details apply to the test as it is usually carried out in this country to determine Westergaard's modulus of subgrade reaction. The procedure is based on that used by the U.S. Corps of Engineers[15].

Apparatus

19·108 The apparatus consists of a bearing plate, equipment to apply a load to the plate, and instruments to measure that load and the settlement of the plate.

19·109 Bearing plates. The usual standard is a circular plate of 30-in. diameter, although smaller plates are often used for tests where less accuracy is required. At the Road Research Laboratory the 30-in. plate is of mild steel and $\frac{5}{8}$ in. thick, and is stiffened with plates of 26-in. and 22-in. diameter and of the same thickness placed on top of it.

19·110 Application and measurement of load. Since mobility is an asset, the most useful form of dead load is a heavy lorry or trailer. Gantry structures loaded with pig iron, concrete blocks or other dead weight are occasionally used and where possible, the gantry may run along the forms used in the construction of the concrete road. It is important that the plate should be at least 8 ft from the nearest support of the dead load.

19·111 The load is transmitted to the plate by a screw or hydraulic jack acting against the underside of the loaded lorry or trailer. With a hydraulic jack the load can be measured by a pressure gauge connected to the output end of the pump and this pump can be separated from the jack by a length of pressure hose, thus enabling the operator to stand clear of the plate. This arrangement is the most convenient for measuring the modulus of subgrade reaction, although the use of an independent load-measuring device, such as a proving ring or an enclosed pressure cell, gives greater accuracy and such a device is always necessary with a screw jack.

19·112 MEASUREMENT OF SETTLEMENT. An independent datum for the measurement of the settlement of the plate is provided by a frame whose supports rest on the ground at points unaffected by the settlement of the plate or the wheels of the vehicle providing the dead load. These supports should usually be at least 8 ft from the plate and from the wheels. The settlement of the plate is taken as the average of the readings of a number of dial gauges arranged symmetrically around the periphery of the plate, these dial gauges being attached to arms fixed to the datum frame.

Test Procedure

19·113 Two alternative test procedures are followed at the Road Research Laboratory. More accurate tests are made with a 30-in. plate, a 30-ton loaded trailer to provide the load reaction, a proving ring to measure the load and four dial gauges to measure the settlement (Plate 19·6A). Rapid, less accurate tests are made with plates of 18-in. or even 12-in. diameter with a mobile laboratory to provide the load reactions, a pressure gauge to measure the load and only two dial gauges to measure settlement (Plate 19·6B). In the latter type of test the value of the modulus of subgrade reaction obtained must be corrected to that for a 30-in. plate as explained later.

19·114 PREPARATION OF TEST AREA. For design purposes the conditions of the moisture content and dry density of the test area ought to be those which are likely to exist when the subgrade has reached a state of relative equilibrium subsequent to the construction of the road. Where such tests are not possible, tests can be made on an area where the required degree of compaction is obtained by hand-tamping in thin layers. If the modulus of subgrade reaction of the natural soil is required, it is usual to remove the top 9 in. of the soil before testing.

19·115 Great care should be taken to seat the plate accurately. The test area is first levelled as much as possible with a trowel or spatula. On fine-grained soils the plate, with its lower surface oiled, is placed on this area and rotated. When the plate is removed the irregularities in the surface, being marked with oil, are trimmed off. This process is repeated until the plate is in contact with the soil over all its area. On more coarsely-grained soils which are difficult to level accurately, or when a quick test procedure is required, the plate can be seated on a layer of fine dry sand in no place thicker than ¼ in. On very stony soils even this is not satisfactory and a flat bearing surface is best obtained with plaster of Paris which can be levelled with the plate before the plaster has set. The bearing test cannot be started until the plaster is sufficiently hardened.

19·116 LOADING PROCEDURE. The plate is first seated by applying a load equivalent to a pressure of 1 lb./sq.in. and releasing it after a few seconds. A load sufficient to cause approximately a 0·01-in. settlement is applied and when there is no perceptible increase in settlement or, in the case of clay soils, when the rate of increase in settlement is less than 0·001 in./min., the average of the readings of the settlement dial gauges is noted. The load as measured by the pressure gauge attached to the jack or by the proving ring is noted both immediately before and after the settlement readings. The load is increased until there is an additional settlement of approximately 0·01 in. and the load and settlement again noted when there is no perceptible increase in settlement. This procedure is repeated until a total settlement of not less than 0·07 in. has been produced.

19·117 U.S. CORPS OF ENGINEERS REVISED LOADING PROCEDURE. In the latest edition of their manual[16], the U.S. Corps of Engineers give details of a more rapid procedure than that given above. The plate is seated in the same manner as before but the actual loading test is restricted to the application of one load only. A load of 7,070 lb. (equivalent to 10 lb./sq.in. for a 30-in. plate) is applied in 10 seconds and held until there is no increase in settlement or, in the case of clay soils, until the rate of increase is less than 0·002 in./min., when the dial gauges for recording the settlement are read.

Results

19·118 Fig. 19·19 shows how the results from a typical plate-bearing test may be recorded and how the average pressure under the plate is plotted against the average settlement.

19·119 EVALUATION OF THE MODULUS OF SUBGRADE REACTION. If Westergaard's assumption that the reaction of the subgrade is proportional to the deflection were entirely correct, the curve in Fig. 19·19 would be a straight line and the slope of this line would be the modulus of subgrade reaction, k lb./sq. in./in. The results, however, usually give a curve which is convex upwards and which has no straight portion even initially; k is therefore usually taken as the slope of the line passing through the origin and the point on the curve corresponding to 0·05-in. settlement, i.e.

$$k = p/0{\cdot}05 \qquad \text{(lb./sq.in./in).}$$

where p = the pressure (lb./sq.in). to cause 0·05-in. settlement

If the revised U.S. Corps of Engineers procedure is used, only the settlement corresponding to a pressure of 10 lb./sq.in. is determined, so that it is impossible to plot a curve of pressure against settlement and k is simply given by:—

$$k = 10/d \qquad \text{(lb./sq.in./in).}$$

where d = settlement (in).

The two procedures will only give identical results when k = 200 lb./sq.in./in. which is an average value for road subgrades.

PLATE – BEARING TEST RECORD

DATE 30/8/47 LOCATION R.R.L. Test Section SITE No. 1
AREA OF PLATE 706·8 sq. in. FACTOR OF PROVING RING 9·562 lb/[illegible]

Proving ring deflection (in)	Load (lb)	Mean bearing pressure (lb/sq in)	Dial gauge readings (in)				Settlement $\left(\frac{1}{1,000}\text{ in}\right)$				Mean settlement $\left(\frac{1}{1000}\text{ in.}\right)$
			1	2	3	4	1	2	3	4	
0·0600	0	0	0·635	0·621	0·66[illegible]	0·674	0	0	0	0	0
0·0811 0·0810	2013	2·85	0·629	0·611	0·656	0·668	6	10	9	6	7¾
0·1011 0·1009	3920	5·54	0·617	0·597	0·641	0·657	18	24	26	17	21¼
0·1121 0·1120	4881	6·90	0·609	0·590	0·635	0·649	26	31	34	25	29
0·1209 0·1208	5818	8·23	0·599	0·581	0·624	0·637	36	40	43	37	39
0·1321 0·1321	6894	9·15	0·590	0·574	0·613	0·623	45	47	54	51	49¼
0·1417 0·1415	7803	11·03	0·581	0·563	0·601	0·619	54	58	66	55	58¼
0·1501 0·1499	8606	12·17	0·574	0·555	0·590	0·610	61	66	77	64	67
0·0600	0	0	0·604	0·588	0·624	0·636	31	33	43	38	36¼

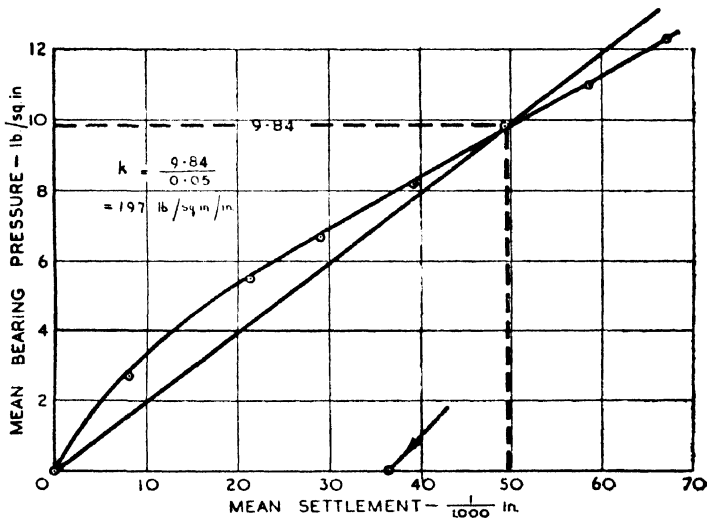



FIG. 19·19 TYPICAL RESULT OF PLATE-BEARING TEST

19·120 ALLOWANCE FOR THE WORST POSSIBLE MOISTURE CONDITIONS OF THE SUBGRADE. The moisture content of the subgrade at the time of the plate-bearing test may increase after the road has been constructed, and the worst possible moisture conditions are sometimes covered by attempting to convert the value of k obtained from the bearing test to a value for the subgrade when soaked. It is impracticable to do this directly by artificially wetting the test area so a secondary soil test is used to convert to the k value for a soaked subgrade. The secondary test used by the U.S. Corps of Engineers is the ordinary consolidation test which they prefer to such tests as a shear test since movements in the soil under the plate are more like the small movements in a consolidation test than the large deformations and shear failure in shear tests. Two series of consolidation tests are made on representative samples of the subgrade, one on the soil in its natural condition of moisture content and dry density and one after the soil has been soaked under a surcharge of 5 lb./sq.in. The results of these tests are plotted as pressure against final deformation when consolidation is complete, as shown in Fig. 19·20. If p is the pressure required in the plate-bearing test to cause a settlement of 0·05 in. and p_s is the pressure required in the consolidation test on the soaked specimens to cause a deformation equal to that produced by pressure p in the consolidation tests on the unsoaked specimens, then the value, k_s, of the modulus of subgrade reaction allowing for soaking is obtained from the equation:—

$$k_s = k \cdot \frac{p_s}{p}$$

Triaxial compression tests not continued to failure but used to determine the modulus of deformation or elasticity of the soil are perhaps a more logical form of secondary test to use to convert the value of k obtained to that for soil with a different moisture content, because Westergaard's modulus of subgrade reaction is largely a measure of the deformation characteristics of the soil.

19·121 CORRECTION WHEN USING PLATES OF SMALLER DIAMETER THAN 30 IN. Assuming the subgrade to be a uniform elastic medium, theoretical relationship can be calculated between the modulus of subgrade reaction and the plate diameter. This relationship is given by the dotted line in Fig. 20·17 in which k for a plate of any diameter, expressed as a percentage of the equivalent value for a 30-in. plate, is plotted against the plate diameter. In practice the subgrade is not uniform and often increases in strength and rigidity with depth and, since the larger the diameter of the plate the greater the depth to which the soil is stressed, this increase in rigidity makes the soil in practice more resistant to deformation than in theory. This is the probable explanation of the difference between the empirical curve of Stratton[17] (the full line in Fig. 20·17) and the theoretical curve for plates of diameter over 30 in.

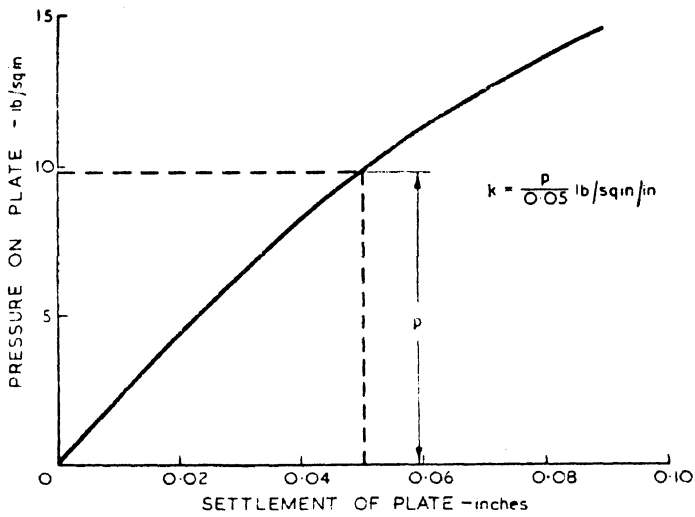
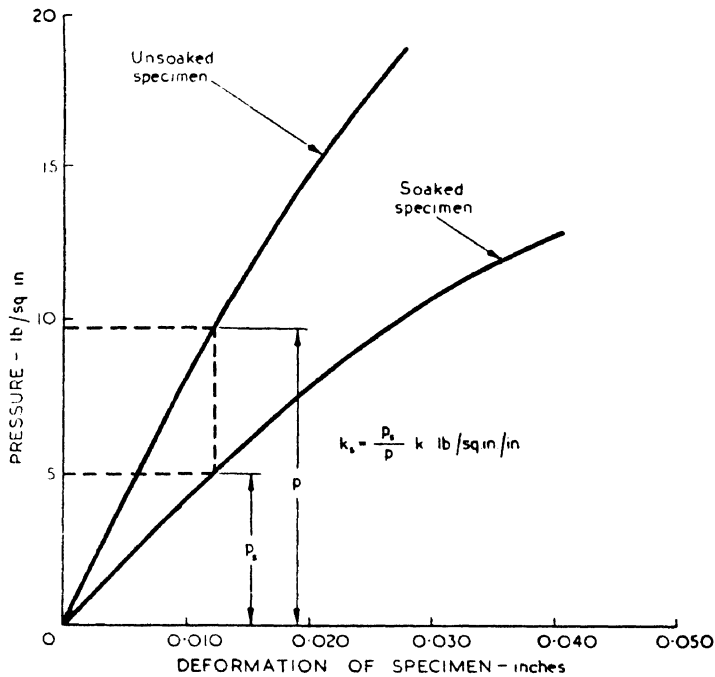
19·122 Stratton's empirical curve was obtained from tests on three different sites and is often assumed to apply to all soil types so that it may be used to determine a correction factor when k is obtained from tests with plates of diameter less than 30 in. For plates of diameter greater than 30 in. the empirical curve shows that there is little variation in k from the value for a 30-in. plate.

(a) Result of plate-bearing test on natural subgrade

(b) Pressure/deformation results from consolidation test

FIG. 19·20 METHOD OF CORRECTING k VALUES FOR SOAKING OF SUBGRADE

Other Types of Bearing Test

19·123 Plate-bearing tests are used in connexion with the construction of roads and runways for other purposes than to determine Westergaard's modulus of subgrade reaction. Some methods of flexible and rigid pavement design use the test to determine the modulus of deformation or elasticity of the subgrade *in situ*, whilst others use the test to determine the bearing capacity of the subgrade for a given deformation. Plate-bearing tests are also made on top of the surfacing or base in order to study the stability of existing rigid or flexible pavements by loading a plate of the same diameter as the tyre/pavement contact-area to an average pressure equal to the contact pressure under the loaded wheel being considered. Such tests may well include the repetitional loading to simulate the effect of traffic. Generally speaking, the test procedure and size of plate used in tests other than those to determine the modulus of subgrade reaction are varied to suit individual requirements and no set procedure can be laid down.

19·124 Bearing tests using square or circular bearing areas have been used in foundation design for many years. Such tests may be used in order to determine the ultimate bearing capacity and the load/settlement characteristics of the natural ground, and by making tests with different bearing areas it is possible to interpolate or extrapolate to the proposed area of foundation. Here again, no set test procedure can be laid down. The subject is referred to in more detail in the Civil Engineering Code of Practice for Site Investigations[(18)].

The California Bearing Ratio Test

19·125 The California bearing ratio test, usually abbreviated to C.B.R. test, is an *ad hoc* penetration test, developed by the California State Highway Department[(19)] for the evaluation of subgrade strengths, in which the load required to cause a plunger of standard size to penetrate a specimen of soil at a standard rate is measured either before or after the soil has been soaked for four days. The test has been adopted by the U.S. Corps of Engineers for the design of flexible pavements[(20)] and it is their test procedure which is most generally used, although it differs slightly from the original California procedure. The test is arbitrary in that the results cannot be accurately related to any of the fundamental properties governing soil strength. However, the deformation of the soil specimen being predominantly shear deformation, the California bearing ratio can be regarded as an indirect measure of shear strength.

19·126 The limitations of the C.B.R. test are those of any *ad hoc* test, viz:—

(1) The test procedure must be strictly adhered to, if results are to be comparable with those previously obtained.

(2) The results give an empirical strength number which cannot be directly related to more fundamental properties governing the strength of soils, such as cohesion.

(3) The results only have a direct application to the method of design for which the test was devised.

On the other hand the C.B.R. test is more flexible than many other penetration tests and can be made on nearly all soils ranging from clay to fine gravel.

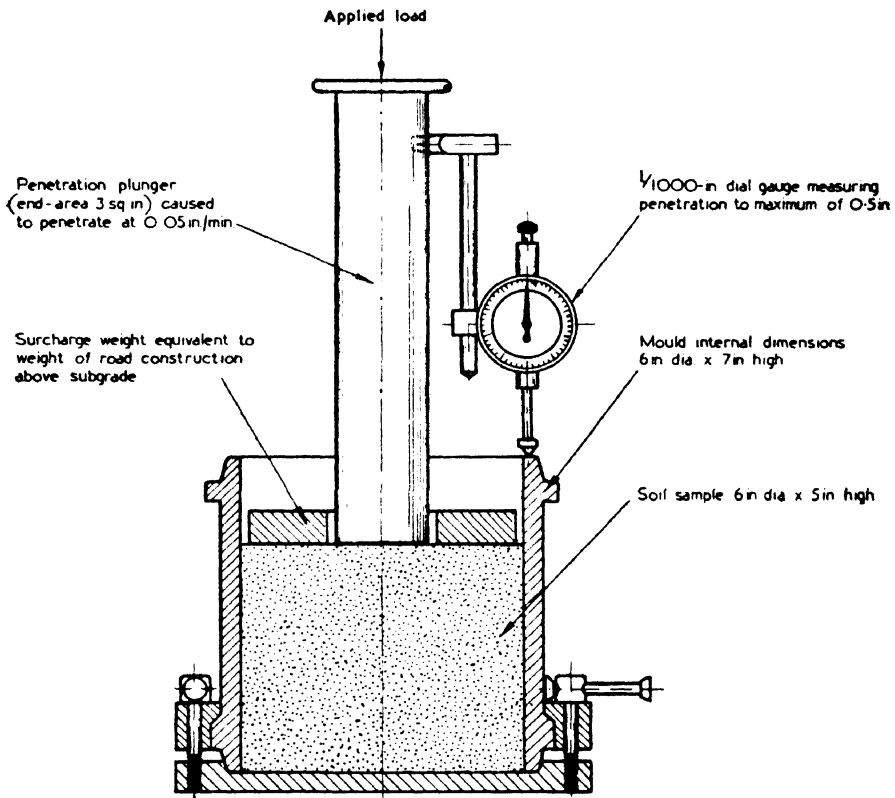



FIG. 19·21 THE CALIFORNIA BEARING RATIO TEST APPARATUS

19·127 Usually the test is made on a specimen of soil contained in a mould of standard size. Such a test is shown diagrammatically in Fig. 19·21. A method of testing *in situ* which has been developed by the U.S. Corps of Engineers is more suitable for some purposes. Investigations to ascertain whether the two forms of test give identical results are still in progress. Because of its wider applicability and use, the test in the mould remains the standard.

Apparatus

19·128 MOULDS. The apparatus required for the laboratory test is shown in Plate 19·7A. The phosphor-bronze mould of internal diameter 6 in. and internal height 7 in. has a detachable perforated base which can be fitted at either end. A bronze displacer disc, 2 in. deep and 6 in. in diameter, enables a specimen exactly 5 in. long to be obtained. A handle which may be screwed into this displacer disc facilitates the removal of the disc. For convenience during the compaction of a soil specimen, a collar 2 in. deep may be fitted to the top of the mould.

19·129 LOADING EQUIPMENT FOR TESTS IN MOULDS. For the penetration test, a testing machine giving a constant rate of strain is preferable. If such a machine is not available a hydraulic testing machine such as that shown in Plate 19·7B

may be used, but with such a machine it is necessary to check the rate of penetration by means of a stop watch. The standard plunger, of circular cross-section 3 sq.in., is placed in the centre of the top of the soil specimen contained in the mould. The dial gauge records the penetration of the plunger during the test, while the testing machine records the load. A similar form of apparatus is a frame and a screw jack on which the correct rate of penetration is obtained by timing and the load on the plunger measured by means of a proving ring. The use of a hydraulic jack, instead of the screw jack, is not recommended because the load tends to be applied as a series of pulses.

19·130 EQUIPMENT FOR THE *in situ* TEST. For an *in situ* test some form of load reaction is required, and a lorry is most suitable for this purpose. Plate 19·8A shows a screw jack fitted on to the back of a mobile laboratory and used to apply the load to the plunger through a proving ring. An independent datum for measurement of penetration is provided by a pipe frame which supports the dial gauges from points on the ground beyond the effect of deformation of the soil under the plunger.

19·131 SOAKING AND SWELL MEASUREMENT. A part of the American standard procedure is to soak the soil specimen before testing and to assist this the base plate is provided with perforations. A check is kept on the swelling of the soil by placing a perforated disc fitted with an extension stem on top of the soil, and recording the movement of the stem by means of a dial gauge supported on the rim of the mould by a tripod.

19·132 SURCHARGE WEIGHTS. In order to simulate the effect of the weight of the layers of pavement above, annular surcharge weights, each of 5 lb., may be placed on the surface of the specimen both during the soaking and during the penetration parts of the tests. These weights may be seen in Plate 19·7A. For the *in situ* test, a plate 10 in. in diameter with a centre hole for the plunger is placed on the soil before the addition of surcharge weights.

Preparation of Specimens

19·133 UNDISTURBED SPECIMENS. If the C.B.R. of a subgrade soil in its existing condition (either natural or compacted) is required, an undisturbed specimen is taken. This is done by fitting to the mould a steel cutting edge of 6-in. internal diameter, and pushing the mould as gently as possible into the ground. This process is facilitated by digging away the soil from the outside as the mould is pushed in. When the mould is sufficiently full of soil, it is removed by under-digging; the top and bottom surfaces are then trimmed flat so as to give a 5-in. length of specimen ready for testing. If the specimen is loose in the mould, the annular cavity should be filled with paraffin wax thus ensuring that the soil receives proper support from the sides of the mould during the penetration test.

19·134 REMOULDED SPECIMENS. If the C.B.R. is required for the soil with a moisture content and density other than that already existing in the field or if it is impossible to obtain undisturbed specimens, the soil may have to be remoulded. The dry density for remoulding should be either the field density or if the subgrade is to be compacted, a value which can be estimated from the results of a B.S. compaction test. If it is proposed to carry out the penetration test on an unsoaked specimen, the moisture content for remoulding should be the same as the equilibrium moisture content which the soil is likely

to reach subsequent to the construction of the road or runway. Assuming a normal type of construction where no water percolates through the surface or from the verges into the top layer of the subgrade, this equilibrium moisture content is likely to be the natural field moisture content which, unless the water-table is near the surface, exists at a depth unaffected by seasonal changes. This depth can be taken as about 4 ft. If it is proposed to carry out the penetration test on a soaked specimen, the moisture content for remoulding should be that which it is estimated the soil will have during construction.

19·135 The material used in the remoulded specimen should all pass a $\frac{3}{4}$-in. B.S. sieve. Allowance for larger material may be made by replacing it by an equal amount of material which passes a $\frac{3}{4}$-in. sieve but is retained on a $\frac{3}{16}$-in. B.S. sieve. This procedure is not satisfactory if the size of the soil particles is predominantly greater than $\frac{3}{4}$ in. The specimen may be compacted statically or dynamically.

19·136 STATICALLY COMPACTED SPECIMENS. By the static method, the desired dry density and moisture content can be obtained directly. The weight of wet soil at the required moisture content to give the intended density when occupying the standard specimen volume is calculated.

Thus, the volume of the C.B.R. specimen

$$= \frac{\pi}{4} \times \frac{6^2}{12^2} \times \frac{5}{12} = 0{\cdot}0818 \text{ cu. ft}$$

If the intended dry density is γ_d lb./cu.ft, the weight of dry soil in the C.B.R. specimen

$$= 0{\cdot}0818\ \gamma_d \times 453{\cdot}6 \text{ gm}$$
$$= 37{\cdot}11\ \gamma_d \text{ gm}$$

If the intended moisture content is m per cent, the weight of wet soil in the C.B.R. specimen

$$= \frac{100 + m}{100} \times 37{\cdot}11\ \gamma_d \text{ gm}$$

19·137 A batch of soil is mixed with water to give the required moisture content. The correct weight of wet soil is placed in the mould and compaction is obtained by pressing in the 2-in. displacer disc, a filter paper being placed between the disc and the soil. The large loads which may be required are developed by a press or testing machine. When the top of the displacer disc is flush with the rim of the mould, the required volume of specimen has been obtained, although with some soil types it is necessary to continue loading until the displacer disc is just below the rim in order to allow for elastic recovery when the load is removed.

19·138 DYNAMICALLY COMPACTED SPECIMENS. An alternative method of preparing the specimens is to compact them dynamically as in the B.S. compaction test. The 2-in. displacer disc is placed at the bottom of the mould and the soil is compacted in layers, each layer being given an equal number of

controlled blows as listed in Table 19·1. The collar must be attached to the top of the mould before compacting the last one or two layers. Three degrees of compaction are obtained by using the three methods given in Table 19·1.

TABLE 19·1

METHODS OF DYNAMIC COMPACTION FOR C.B.R. TEST

Type of compaction	Number of layers	Magnitude of blow		Number of blows per layer
		Weight of hammer (lb.)	Fall (in.)	
Equivalent " 15-blow standard " compaction	3	$5\frac{1}{2}$	12	33
Equivalent B.S.: 1377 compaction	3	$5\frac{1}{2}$	12	55
Equivalent modified A.A.S.H.O. compaction	5	10	18	55

19·139 After the compaction of the top layer the collar is removed and the soil trimmed level with the rim of the mould. The mould is weighed both full and empty so that the bulk density and, knowing the moisture content, the dry density can be calculated.

19·140 The results of tests on specimens compacted at the required moisture content by these three methods of dynamic compaction can be plotted as C.B.R. against dry density so that the C.B.R. for a particular density can be obtained by interpolation.

19·141 RELATIVE MERITS OF STATIC AND DYNAMIC COMPACTION. The static method gives the required dry density directly, whereas the dynamic method involves interpolation. It does however require considerable pressure, and it has the additional disadvantage that, although the mean density of the specimen may be correct, the actual density may vary with depth through the specimen. The variation in dry density through the specimen may be as much as 5 per cent of the mean dry density for static compaction in which the soil is poured into the mould and then compacted, but the variation is reduced if the soil is tamped by hand during the pouring process.

19·142 PREPARATION FOR *in situ* TEST. A circular area about 12 in. in diameter is trimmed flat, special care being taken with the central area on which the plunger will bear. A thin layer of fine sand may be used to seat the plate on which the surcharge weights rest, but the use of sand to seat the plunger itself should be avoided. If it is impossible to trim the soil sufficiently to obtain good seating of the plunger a thin layer of plaster of Paris may be used, care being taken to remove any plaster of Paris extending beyond the area of the plunger.

Test Procedure

19·143 The following procedure, except for the soaking, applies to *in situ* tests as well as to laboratory tests on undisturbed and remoulded specimens.

19·144 SURCHARGE. Sufficient surcharge weights are placed on the surface of the soil to equal the actual or estimated weight of construction to be placed on top of the particular layer under consideration. Small errors in the estimation of surcharge pressures are of minor importance except in tests on cohesionless sands. For design purposes, the weight of surcharge used should be not less than 10 lb. for a test in a mould and 30 lb. for an *in situ* test. Each 5-lb. weight is approximately equivalent to 2½ in. of construction when used with a mould, or 1 in. of construction when used with the 10-in. diameter plate in an *in situ* test.

19·145 SEATING PLUNGER. A load of 10 lb. is used to seat the plunger on the surface of the soil, and the load and the penetration gauges for measuring are set to zero. The seating load is neglected in the final calculation of pressures.

19·146 APPLICATION OF LOAD. The load is applied to the plunger so that the rate of penetration remains constant at 0·05 in./min. If the pointer of the dial gauge used to measure the penetration makes one revolution for a movement of 0·05 in. the correct rate of penetration can be obtained by keeping this pointer in step with the second hand of a clock. Readings of the load should be taken at least at the following penetrations:—0·025, 0·05, 0·075, 0·10, 0·15, 0·20, 0·30, 0·40, and 0·50 in.

19·147 MOISTURE CONTENT DETERMINATION. After the penetration test is completed, a sample is taken from the soil immediately under the plunger and a second sample from about 1 in. further into the soil. The average of the two moisture contents obtained from these samples is calculated and used as a check on the moulding moisture content if the specimen is unsoaked or on the degree of saturation if the specimen has been soaked.

19·148 SOAKING. In an attempt to bring the specimen to the worst moisture condition which may exist subsequent to construction (water-table in the surface) the American procedure requires the specimen to be soaked before testing. For this purpose, the specimen is completely immersed so as to allow free access of water to both top and bottom. The perforated disc is placed on the surface of the soil and supports the same number of surcharge weights as that to be used in the penetration test. The stem of the disc is used to measure the swell of the soil by means of the dial gauge and tripod. The soaking is continued for 4 days, after which the specimen is allowed to drain for 15 min. before the penetration test is begun. If the C.B.R. in both the soaked and unsoaked conditions is required, one end can be tested unsoaked, the resulting depression in the surface made up and the other end tested after soaking.

Test Results

19·149 LOAD/PENETRATION CURVE. The readings of load are plotted against the readings of penetration and a smooth curve is drawn through the points. The curve is mainly convex upwards although the initial portion of the curve may be concave upwards; the concavity is assumed to be due to surface irregularities. A correction is applied by drawing a tangent to the curve at the point of greatest slope. The corrected curve is then this tangent plus the convex portion of the original curve with the origin moved to the point where the tangent cuts the horizontal axis. The results of two tests, one requiring correction by this method, are given in Fig. 19·22.

TEST DATA

Penetration (in)	Standard load (lb)	TEST 1			TEST 2		
		Load reading (lb)	Corrected load (lb)	C.B.R (%)	Load reading (lb)	Corrected load (lb)	C.B.R (%)
0·025		710			45		
0·050		1,135			160		
0·075		1,430			360		
0·100	3,000	1,640		(54)	650	1,140	38
0·150		1,905			1,175		
0·200	4,500	2,110		47	1,605	1,800	(41)
0·300		2,375			2,090		
0·400		2,585			2,380		
0·500		2,690			2,595		

Ringed numbers indicate C.B.R values to be taken for design purposes

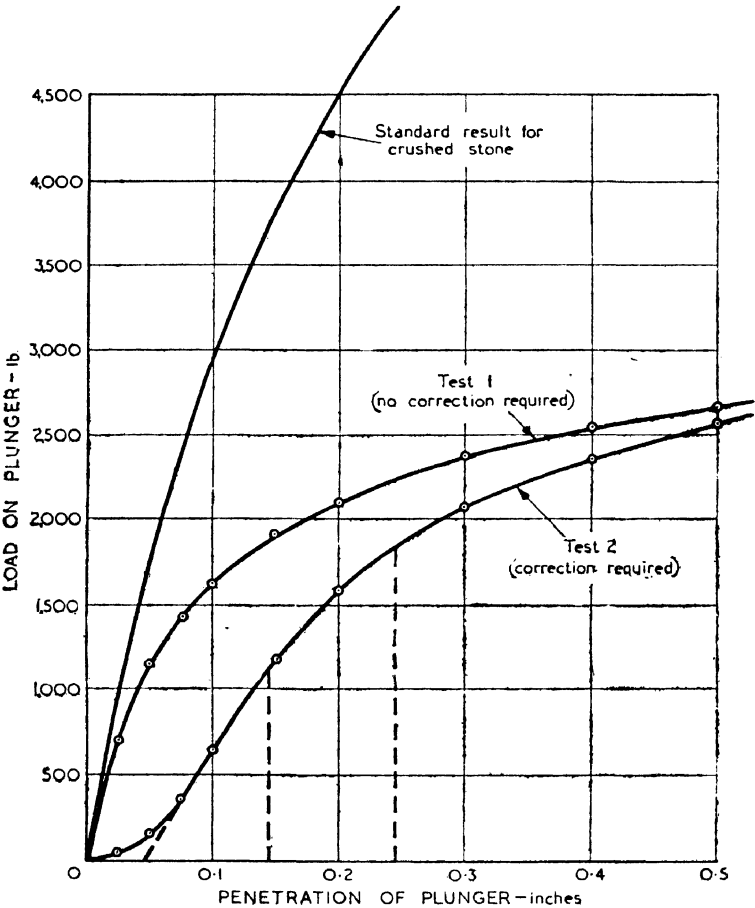



FIG. 19·22 TYPICAL RESULTS OF THE C.B.R. TEST

CALIFORNIA BEARING RATIO TEST

Date 24th Jan. 1949 Location Ripon Soil from subgrade No 1 (top of specimen)

Soil type Silty sand

Dry density 120 lb/cu ft

Tin No	Moisture content (per cent)	Mean moisture content (per cent)
A 63	11·5	11·4
A 71	11·3	

Bearing value 30% at 0·1 in penetration, no correction necessary

Penetration of plunger (inches)	Load on plunger (lb)	Penetration of plunger (inches)	Load on plunger (lb)	Penetration of plunger (inches)	Load on plunger (lb)
0·010	120	0·110	955	0·225	1,470
0·020	230	0·120	1,005	0·250	1,560
0·030	355	0·130	1,050	0·275	
0·040	490	0·140	1,100	0·300	
0·050	600	0·150	1,140	0·325	
0·060	690	0·160	1,200	0·350	
0·070	750	0·170	1,240	0·375	
0·080	805	0·180	1,280	0·400	
0·090	860	0·190	1,320	0·450	
0·100	915	0·200	1,370	0·500	

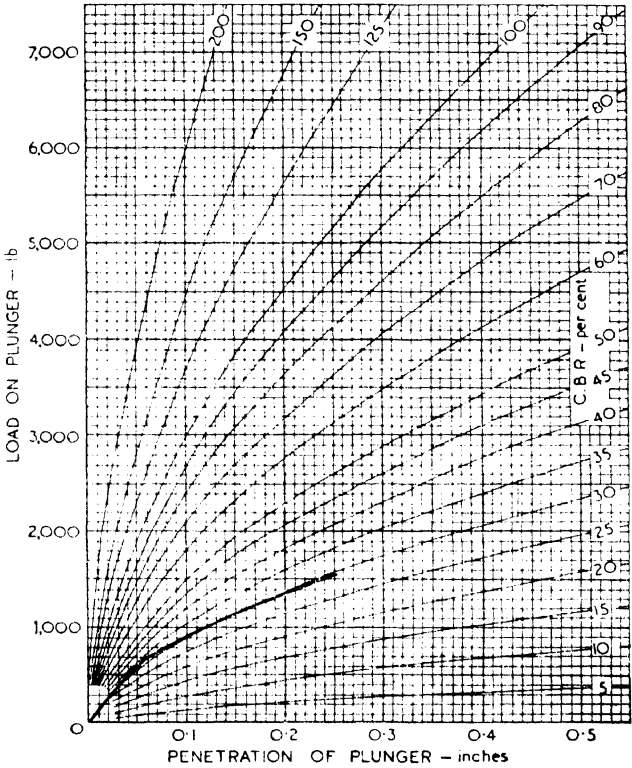



FIG. 19·23 C.B.R. PLOTTING SHEET WITH TYPICAL RESULT

19·150 CALCULATION OF C.B.R. The California bearing ratio is an arbitrary coefficient which attempts to express the information obtainable from the load/penetration curve as a single quantity. The values of the load at the corrected penetrations of 0·1 in. and 0·2 in. are expressed as percentages of the standard loads of 3,000 lb. and 4,500 lb. respectively. These standard loads were obtained from tests on a crushed stone which was defined as having a California bearing ratio of 100 per cent. Usually the value at 0·1-in. penetration is greater than that at 0·2-in. penetration and the former is taken as the California bearing ratio for design purposes. However, if the 0·2-in. value is the greater, the test is repeated on a fresh sample and if this check test gives a similar result the 0·2-in. value is taken for design purposes.

19·151 The results can be conveniently plotted on a chart of the type shown in Fig. 19·23 from which the C.B.R. at any penetration can be read off directly. If the curve requires correction, the corrected C.B.R. value can be obtained by placing a mask consisting of a tracing of the chart on top of the plotted curve with its origin coinciding with the point where the tangent cuts the horizontal axis as already described.

19·152 RELATIONSHIP BETWEEN C.B.R., DRY DENSITY AND MOISTURE CONTENT. An investigation has been made at the Road Research Laboratory to obtain the relationships between C.B.R., moisture content and dry density for several different soils. Results for five soils at the respective maximum dry densities given by the B.S. compaction test for a range of moisture contents above and below the optimum moisture content are given in Fig. 19·24. In the case of the granular soils containing fines, the sharp decrease in C.B.R. which occurred with increasing moisture content is thought to be due to the fines acting as a binding agent at low moisture contents and as a lubricant at high moisture contents. The C.B.R. of the soils was found to increase with an increase in dry density but, for the usual range of variation of dry density and moisture content, the effect of the latter on the C.B.R. was found to be larger.

The North Dakota Cone Test

19·153 The North Dakota State Highway Department[21] has developed a cone penetration test for use with a pavement design method similar to that associated with the California bearing ratio. The test employs a simple apparatus which can easily be used for *in situ* field tests. The test is simpler and more rapid than the C.B.R. test, it may be made on the subgrade either in its natural state or after it has been prepared by compaction or stabilization, and it can be applied directly in an empirical method of pavement design. Although originally designed for *in situ* tests in the field the test can also be made on soil remoulded in large moulds such as the C.B.R. mould. The main disadvantage of the test is that its use is restricted to fine-grained soils and the test is probably only reliable for clayey soils. In common with other cone penetration tests, the presence of even small amounts of stone in an otherwise fine-grained soil can make the results quite unreliable.

Apparatus

19·154 The penetrometer, shown in Plate 19·8B, consists essentially of a shaft with a sharp cone (half angle, 7 degrees 45 minutes) attached to one end. The movement of the shaft relative to its supporting frame is measured by means

Soil	Maximum dry density (lb./cu.ft)	Optimum moisture content (per cent)
Sand-gravel with fines	129	8
Coarse sand with fines	125	10
Clean fine sand	102	13
Sandy clay	112	16
Heavy clay	97	26

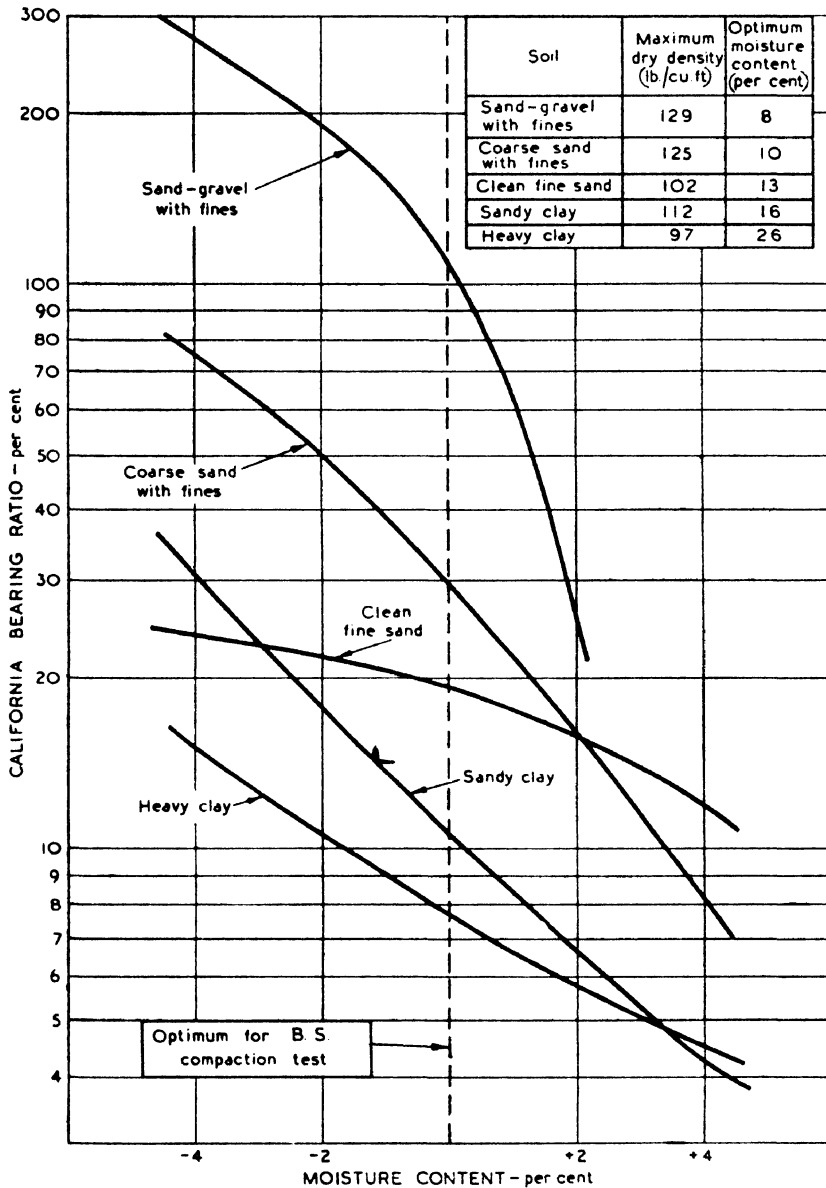



FIG. 19·24 RELATIONSHIP BETWEEN C.B.R. AND MOISTURE CONTENT FOR SOILS AT THE MAXIMUM DRY DENSITY GIVEN BY THE B.S. COMPACTION TEST

(Specimens statically compacted and tested unsoaked without surcharge)

of a vernier scale and the shaft can be locked to the frame when necessary. The cone can be loaded during the test by placing weights on a disc fixed to the top of the shaft.

Test Procedure

19·155 The test is made directly on the subgrade, an area of which is trimmed level for the purpose. When the penetrometer has been placed in position, the cone is moved down until it just touches the surface of the subgrade, and the shaft is locked in position. The cone is then loaded to 10 lb., after which the shaft is unlocked for 1 minute. The shaft is again locked and the readings of the cone penetration noted; the load on the cone is then increased to 20 lb. and the cone again allowed to penetrate for 1 minute. This procedure is repeated for loads of 40 lb. and 80 lb. All loads include the weight of the cone and shaft.

Results

19·156 The results are expressed as a bearing pressure, taken as the load divided by the cross-sectional area of the cone at surface level. Theoretically the bearing pressure should be the same for each load, which means that the penetration for the 20-lb. load should be half that for the 80-lb. load, but in practice this is not so because of zero errors due to the point of the cone being rounded. For this reason, the reading obtained with the 10-lb. load is rejected and a correction C is added to the remaining penetration readings, as shown in Table 19·2.

TABLE 19·2

TYPICAL RESULT OF NORTH DAKOTA CONE TEST

Bearing area = π (penetration x tan 7° 45′)^2

Load (lb.)	Penetration reading (in.)	Corrected penetration (in.)	Bearing value (lb./sq. in.)
10	0.94	1.18	—
20	1.36	1.60	135
40	2.07	2.31	129
80	2.96	3.20	135
Mean bearing value - lb./sq.in.			131

Correction C to be added to all penetration readings is given by

$$C = \rho_{80} - 2\rho_{20}$$

where ρ_{80} = Penetration at 80-lb. load

ρ_{20} = Penetration at 20-lb. load

$$\therefore \quad C = 2{\cdot}96 - 2 \times 1{\cdot}36 = 0{\cdot}24 \text{ in.}$$

Other Types of Penetration Test

19·157 During the site exploration for a foundation which will stress the soil to a considerable depth, it is frequently uneconomical to make the large number of deep boreholes necessary to obtain an accurate picture of the horizontal

and vertical variations in soil type and soil strength. Deep penetration tests are often made as a rapid means of supplementing the information from a few boreholes. They can also be used to estimate the resistance to driving of bearing piles.

19·158 Many forms of apparatus have been used, from steel rails to specially designed probes[2], either jacked into the soil or driven by a hammer. Measurements are made of the resistance to penetration as the penetration increases so that a continuous record to any depth is obtained. It is usually necessary to measure the direct toe resistance and the skin or frictional resistance separately and the direct toe resistance must be correlated with the results of laboratory shear tests made on undisturbed samples taken from the boreholes.

19·159 A form of penetration test that is in common use for foundation work on the Continent is the cone penetration test developed by the Swedish and Danish State Railways[22][23]. The results of this test are expressed as a firmness index which must be correlated with the shear strength of the particular soil by means of laboratory tests. Another form of penetration test often used as a control test for the density of compacted subgrades of roads and airfields is the Proctor penetration needle test described in Chapter 9.

Applications of Bearing and Penetration Tests

19·160 The following are the principal uses of bearing and penetration tests:—

(1) The calculation of the stresses within concrete pavements by means of Westergaard's analysis. (Plate-bearing test.)

(2) The design of road pavements with respect to subgrade and often sub-base strength. (C.B.R., plate-bearing test, North Dakota cone, etc.)

(3) The testing of the stability of existing road and airfield pavements, (Plate-bearing tests.)

(4) The prediction of settlements and bearing capacities under embankments and building foundations. (Bearing and penetration tests.)

(5) The determination of the elastic or deformation properties of soils *in situ*. (Plate-bearing and other bearing tests.)

APPENDIX TO CHAPTER 19

Construction of Masks for the Unconfined Compression Test Machine

Assumptions

19·161 It is assumed that the soil specimen does not change in volume during loading and that its shape remains a cylinder.

Method

19·162 The mask is constructed by plotting lines of constant stress on a graph which has a vertical straight-line axis of load and a curved axis of deformation corresponding to those of the chart. These lines and their axis are then traced on to transparent material so that the mask can be superimposed on any chart. The calculations for the line of stress σ lb./sq.in. are given below.

Ordinate at Zero Deformation

19·163 Stress on specimen = σ lb./sq.in.

Cross-sectional area of specimen = $\frac{\pi}{4}(1\frac{1}{2})^2$ = 1·767 sq.in.

∴ Load on specimen = 1·767 σ lb.

If stiffness of spring = λ lb./in. (= load in lb. to cause 1-in. extension) then

extension of spring to produce stress σ

$$= \frac{1\cdot767\,\sigma}{\lambda} \text{ in.} \quad \ldots\ldots\ldots\ldots \quad (2)$$

This extension is the ordinate of the curve of stress σ lb./sq.in. at zero deformation.

Ordinate at Deformation d in.

19·164 Original length of specimen = $3\frac{5}{8}$ in. overall and 3 in. to apices of cones.

Length of cylinder of same volume and diameter = $3\frac{1}{4}$ in.

If volume of specimen before loading = volume of specimen after loading,

$3\frac{1}{4} \times 1\cdot767 = (3\frac{1}{4} - d)A$ cu.in.

where A is the new cross-sectional area.

But load on spring to produce stress σ = σA

$$\therefore \quad \text{Extension of spring} = \frac{\sigma A}{\lambda} = \frac{1\cdot767\,\sigma\,3\frac{1}{4}}{\lambda\,(3\frac{1}{4}-d)} \text{ in.} \quad \ldots\ldots\ldots \quad (3)$$

This extension is the ordinate of the curve at deformation d.

19·165 Calculations can be made from equations (2) and (3) for various stresses at various deformations and from these results the lines of various constant stresses can be drawn.

SUMMARY

19·166 This chapter discusses the properties governing the strength of soil and describes the more common methods of measuring soil strength. It is divided into three sections:

19·167 The first section outlines the characteristics of soil as an engineering material: Coulomb's empirical law relating the shear strength to the apparent cohesion and the angle of shearing resistance, the criterion of failure employing the true cohesion and true angle of internal friction, the behaviour of soil under stresses less than those required to cause failure, the types of test used to measure the properties governing the strength of soils and the factors affecting the results of strength tests.

19·168 The second section deals with shear tests. Details are given of the usual types of shear box apparatus and triaxial compression apparatus, the various test procedures—immediate, consolidated-quick and slow tests—and the calculation of the angle of shearing resistance, the apparent cohesion and

the modulus of deformation from the results obtained. Details are also given of the unconfined compression apparatus, test procedure and calculation of the apparent cohesion of a clay when tested under conditions of zero angle of shearing resistance. Other types of shear test are also mentioned, including a field test for clays known as the vane test.

19·169 The third section deals with bearing and penetration tests. Details are first given of the apparatus required for plate-bearing tests to determine Westergaard's modulus of subgrade reaction: both rapid and more accurate procedures for such tests, the calculation of the modulus of subgrade reaction from the results and the corrections for a saturated soil and for plate diameters less than 30 in. Mention is also made of other types of bearing test. The next part of the section deals with a penetration test developed for a particular method of pavement design and known as the California bearing ratio test: the apparatus for such tests in moulds and *in situ* undisturbed sampling, and both static and dynamic methods of remoulding specimens, test procedure based on that developed by the U.S. Corps of Engineers and the interpretation of the test results. This is followed by a brief description of the North Dakota cone test, a penetration test associated with a pavement design method similar to the California bearing ratio method, and other forms of penetration test are also discussed.

19·170 An appendix on the construction of masks for the unconfined compression test is included.

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CHAPTER 20

PAVEMENT DESIGN

INTRODUCTION AND GENERAL OUTLINE

The Function of Pavement Design

20·1 A natural earth track can neither support modern wheel loads nor provide an adequate wearing surface, therefore a constructed pavement is required on top of the soil in order to distribute the wheel load sufficiently and to provide the necessary wearing surface. It is also important that the construction should be such that no part of the pavement is overstressed. Thus the function of pavement design is to evaluate the type, thickness and treatment of materials that will most economically fulfil these requirements.

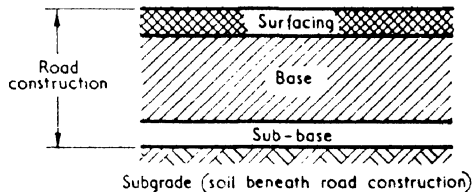



FIG. 20·1 BASIC STRUCTURAL ELEMENTS OF A PAVEMENT

The Structural Components of a Road

20·2 The road is usually built up in several layers as shown in Fig. 20·1, each layer having a special function. In a concrete road, the concrete slab usually provides the wearing surface as well as distributing the load (Fig. 20·2(a)). On poor subgrades, a sub-base of compacted gravel or stabilized soil is required underneath to distribute the load further and to provide a satisfactory surface on which to construct the slab (Fig. 20·2(b)). A heavier construction is shown in Fig. 20·2(c) in which the wearing surface is provided by a bituminous carpet the concrete slab then becomes the base and only has to distribute the load.

20·3 In a bituminous road the load is largely distributed by the base, the chief function of the surfacing being to provide a wearing surface (Fig. 20·3(a)). On some subgrades, particularly clay subgrades a compacted stone or stabilized soil sub-base may be required under the base in order to distribute the load further and to prevent the clay working up into the base (Fig. 20·3(b)).

20·4 However much the load is distributed by the surface, base and sub-base, the subgrade must eventually carry the whole load due to the road structure and the traffic upon it. The strength of the top of the subgrade is therefore frequently increased by compaction and occasionally by stabilization. The top layer of subgrade thus treated may be considered as an additional sub-base.

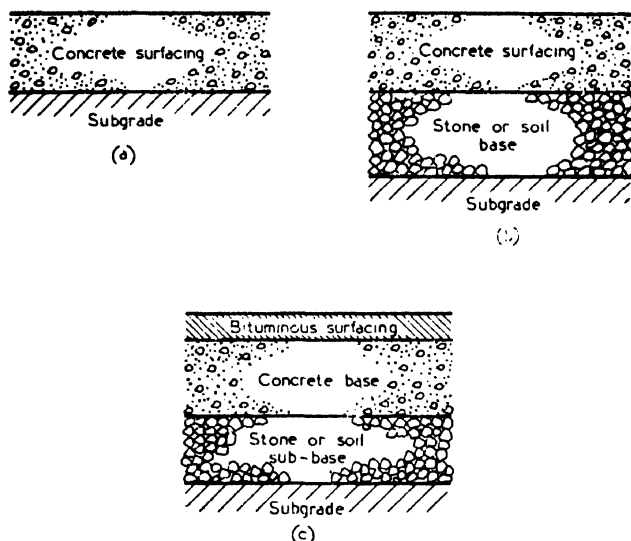



FIG. 20·2 TYPICAL FORMS OF ROAD CONSTRUCTION: CONCRETE

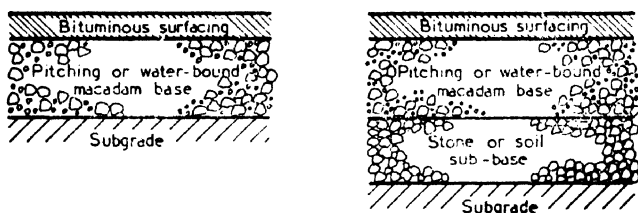



FIG. 20·3 TYPICAL FORMS OF ROAD CONSTRUCTION: BITUMINOUS

Causes of Structural Failure

20·5 EXCESSIVE STRESSES DUE TO TRAFFIC. These may arise in the subgrade leading to an increased deformation with each application of load with consequent failure of the layers above; this is probably the most important cause of failure and is the only one considered in many methods of pavement design. Excessive stresses in the base or sub-base may lead to failure of the road even though the subgrade does not become excessively deformed.

20·6 EXCESSIVE STRESSES DUE TO OTHER CAUSES. Excessive stresses in the surfacing or base may be produced by secondary causes such as temperature or moisture changes. These are only considered in concrete roads.

20·7 STRIPPING OF THE SURFACING. Stripping of the surfacing may occur due to insufficient strength of the adhesive bond either between the individual particles of mineral aggregate, or between the surfacing and the base; this is largely confined to bituminous surfacings and is not further discussed here.

Changes in the Subgrade

20·8 The strength of the subgrade at the time the road is constructed will not necessarily remain the same throughout its life: it may be higher at times and

will almost certainly be lower at other times. These changes in the strength of the subgrade are known as regression and may be due to any of the following causes:—

20·9 ACTION OF TRAFFIC. Repeated wheel loads can cause the compaction and consolidation of the subgrade, leaving localized areas of the base unsupported, and producing in the pavement stresses whose magnitude cannot be calculated and which may be especially serious in concrete roads.

20·10 INCREASE OF MOISTURE CONTENT. This is the most important cause of regression. For a given dry density, nearly all soils decrease in strength with increasing moisture content and this effect is most marked in clayey soils. Any design should therefore take account of this effect, either by using the soil strength measured when the soil is at the worst moisture condition likely to arise, or by providing adequate protection of the subgrade and ensuring that the pavement is impervious where this is feasible. Subgrade drainage does not necessarily prevent an increase of moisture content which may occur by the migration of water from the water-table.

20·11 ACTION OF FROST. When the temperature of the soil falls below the freezing point, ice lenses may form in the soil and cause damage to the pavement. Frost can also be an indirect cause of regression of the subgrade when it causes cracks to form in the surfacing, thus permitting water to penetrate to the base and subgrade.

20·12 Frost-susceptible soils such as chalk and limestone normally form strong subgrades. The depth of frost penetration estimated from records of soil temperature during severe winters must, however, be considered in estimating the required thickness of construction in addition to the strength of the subgrade.

20·13 DECREASE IN MOISTURE CONTENT. The strength of the subgrade increases with decreasing moisture content, but in clay soils this may involve considerable shrinkage of the soil if the decrease in moisture content is large. After a long drought such as occurred during the summer of 1947, the subgrade near the verge dries out more than the subgrade under the crown of the road. This can cause severe differential movement of the slabs in a concrete road and severe longitudinal cracks in a tarmacadam road. Cracks up to 3 in. wide recently observed in a class A road were due to differential shrinkage of the clay subgrade.

The Necessity for using a Design method

20·14 In the past, when no design method was employed, much the same thickness of construction was sometimes placed on a weak clay as on a strong gravel subgrade. As the subgrade strengths common in this country vary over a range of at least 25 to 1, a reliable method is needed to evaluate the economic thickness and type of construction. Investigations of road failures have shown that insufficiently thick pavements have often been laid on weak subgrades. Similarly, it is known that unnecessarily thick pavements have been laid on strong subgrades. A small expenditure on preliminary soil testing and the intelligent use of design methods should lead to fewer road failures and lower costs of construction.

Types of Pavement Construction

20·15 Most methods of pavement design postulate either a rigid or a flexible type of pavement.

20·16 RIGID PAVEMENTS. Concrete is the only important form of construction of this type. Cement-stabilized soil construction might perhaps be considered as intermediate between rigid and flexible. Tensile stresses as high as 700 lb./sq.in. can develop in concrete pavements which are thus able to bridge small weaknesses and depressions in the subgrade.

20·17 FLEXIBLE PAVEMENTS. Flexible pavements generally consist of a bitumen or tarmacadam surfacing laid in one or more layers, a compacted gravel, hardcore or pitching base, and possibly a compacted sub-base of low-grade gravel or ashes. The pavement is able to resist only very small tensile stresses and any change in shape of the surface of the subgrade is followed by a corresponding change in the road surface.

Traffic Loads

20·18 MAXIMUM WHEEL LOAD. Unless two wheels are sufficiently closely spaced for the areas of subgrade stressed by each wheel to overlap, it is the maximum wheel load that is important for pavement design, not the gross load of the vehicle. Assuming two-thirds of the weight to act on the rear axle, the maximum wheel load of a 6-ton four-wheel lorry is 2 tons and it is this 2 tons that determines whether a particular pavement can carry the lorry, not the gross load of 6 tons.

20·19 MULTIPLE WHEEL LOADS. For flexible pavements, Spangler(1) proved theoretically that dual wheels at 10-in. centres, each with a load of 4,000 lb., were no more severe on a 4-in. flexible pavement than a single wheel load of 4,000 lb., and that a single 8,000-lb. wheel load was considerably more severe.

20·20 For rigid pavements Sparkes(2) evaluated the effect of common arrangements of multiple wheel loads for road vehicles. His results suggest that the effect of multiple wheel loads is to increase the stresses to about 25 per cent above that caused by a single wheel load for critical loading conditions.

20·21 The effect of heavy multiple aircraft wheels cannot be treated so simply. The problem of evaluating the relative effect on a given pavement of different numbers and arrangements of wheels and tyre pressures and wheel spacings for the same gross load on an aircraft undercarriage is still largely unsolved.

20·22 IMPACT. An accurate evaluation of the impact effect on roads is not easy as it depends on wheel load, vehicle speed, vehicle characteristics, tyre type and road irregularity. However, the work of Aughtie, Batson and Brown(3) has shown that even with an artificial irregularity consisting of a plank one inch thick, the increase in load transmitted to the road is unlikely to be greater than 30 per cent of the static load. Modern roads rarely, if ever, have irregularities as severe as this artificial one(4). It is therefore reasonable to make no allowance for impact when designing roads with well finished surfaces and in no case to make an allowance greater than 30 per cent. Furthermore, when wholly empirical methods of pavement design are used, no allowance for impact need ever be made because the thickness of construction is decided from past experience of the thickness required for the particular wheel load and thus impact has already been taken into account.

20·23 REPETITION OF LOADS. The repeated application of loads to a pavement may result in sufficient cumulative permanent deformation to cause failure although a single application of load would not.

20·24 Methods of pavement design that are not wholly empirical rarely take into account repetitions of load or traffic intensity, so that for such methods an allowance should be made by increasing the wheel load before substituting it in the design formulæ. This increase should vary with traffic intensity but accurate evaluation is almost impossible in the present state of knowledge on the subject. As a rough guide the figures in Table 20·1 may be used for either rigid or flexible pavements.

TABLE 20·1

ALLOWANCE FOR REPETITION OF LOADS

Traffic intensity	Increase on static wheel load (%)
Light	0
Medium	10
Heavy	20

In wholly empirical methods of design, allowance for traffic intensity is usually part of the method itself.

20·25 CONTACT PRESSURE AND AREA. The contact pressure between the wheel and the road depends on the tyre inflation pressure and the stiffness of the tyre walls, and is generally taken as 110 per cent of the inflation pressure for aircraft and 130 per cent for road vehicles. However, Markwick and Starks[5] have shown that it may be as great as 150 per cent for road vehicles.

20·26 The contact pressure is usually assumed to be uniformly distributed so that the contact area is the total wheel load divided by the contact pressure. In general, this area is approximately elliptical but no great error is involved if it is considered to be circular, and calculations are much simplified by such a consideration.

Methods of Pavement Design

20·27 There are a great many methods of pavement design, differing considerably in their reliability and in their method of approach to the problem. The problem is too complex and its study of too recent origin for a method to have yet been developed which is as sound, reliable and generally accepted as design methods in other branches of engineering. Most methods consider only some of the causes of failure given in paras. 20·5 to 20·13. Generally speaking, the reliability of any method is proportional to the amount of experience or experimental verification behind the method, and all methods require a considerable amount of common sense and experience on the part of the engineer who applies them. Even though existing methods are so diverse, they can be classified in four distinct groups.

20·28 GROUP A. EMPIRICAL METHODS USING NO SOIL STRENGTH TESTS. The thickness of construction is determined from past experience of the thickness required for similar wheel loads and soils giving similar results in such classification tests as the particle-size analysis and the liquid and plastic limit tests.

20·29 GROUP B. EMPIRICAL METHODS USING A SOIL STRENGTH TEST. This test is commonly a penetration or bearing test and is frequently only applicable to its associated design method. The test is used for comparing past experience of the thickness of construction required on soils with similar strength values.

20·30 GROUP C. METHODS BASED PARTLY ON THEORY AND PARTLY ON EXPERIENCE. The fundamental stress/strain properties of the subgrade soil and sometimes the base material are determined by shear or bearing tests and the results are employed in a simplified or modified theory of stress distribution which has been found to have some experimental justification.

20·31 GROUP D. WHOLLY THEORETICAL METHODS. These are based on a mathematical analysis of the stresses and strains throughout the pavement and subgrade and the true stress/strain characteristics of the various materials. This is an ideal that may never be reached.

20·32 In the ensuing sections each group is dealt with separately in more detail and examples of the commoner design methods are given.

DESIGN METHODS OF GROUP A

EMPIRICAL METHODS USING SOIL CLASSIFICATION TESTS

Introduction

20·33 Provided the base and sub-base are adequately compacted, the major factor in deciding the thickness of construction required is usually the strength of the subgrade. This strength is dependent upon:—

(1) Moisture content of the soil.

(2) Dry density of the soil.

(3) Soil composition and internal structure.

Thus if (1) and (2) are controlled by adequate subgrade drainage and compaction, the strength of the subgrade and hence the thickness of construction depends largely on the composition and structure of the subgrade soil. This argument is the basis of design methods in this group. The methods differ only in the tests used to define composition and structure and in the way the results of these tests are combined to give a measure of the strength of the subgrade or thickness of construction required.

U.S. Highway Engineers' Group Index Method

20·34 A sub-committee of the Highway Research Board of the U.S.A. concerned with the "Classification of Highway Subgrade Materials," on which Steele acted as chairman[6] suggested a design method which is typical of Group A methods. The method is based on the revised U.S. Public Roads Administration classification, described in Chapter 4, which uses an empirical quantity called the "group index." This is claimed to be an inverse measure of the thickness of sub-base required. The thickness of base and surfacing, on the other hand, is varied according to the volume of commercial traffic expected. The higher the "group index" of the subgrade the lower its strength and the greater the thickness of sub-base required.

20·35 The formula for the group index is as follows:—

$$\text{Group Index} = 0{\cdot}2a + 0{\cdot}005ac + 0{\cdot}01bd \quad \dots \quad (1)$$

where a = That portion of percentage of subgrade soil passing No. 200 sieve* greater than 35 and not exceeding 75, expressed as a positive whole number (0 to 40).

b = That portion of percentage of subgrade soil passing No. 200 sieve* greater than 15 and not exceeding 55, expressed as a positive whole number (0 to 40).

c = That portion of the numerical liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (0 to 20).

d = That portion of the numerical plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (0 to 20).

The group index of a soil can be more easily calculated from the charts given in Fig. 4·3.

20·36 The soil classification table is given in Table 4·5. The tentative design curves suggested by Steele are given in Fig. 20·4 and worked examples in Fig. 20·5. These design curves relate the thickness of sub-base directly to the group index of the subgrade (except that a well graded gravel subgrade, classification A-1-a, is considered to be stronger than any other soil having a zero group index and so the thickness of sub-base is reduced accordingly). It should be noted that these design curves were presented as only approximate and with the recommendation that each user of the method should modify them to suit his own conditions as experience dictated.

20·37 The design curves are based on the following assumptions with regard to compaction and drainage:—

(1) Compaction of the subgrade to be not less than 95 per cent of the maximum dry density determined by the standard A.A.S.H.O. test†, and compaction of the sub-base and base not less than 100 per cent.

(2) The subgrade to be sufficiently above the water-table to permit the proper compaction of the subgrade prior to placing the base or sub-base, and soil drainage or sufficient embankment height to be provided where necessary to keep the water-table at least 3 or 4 ft below the road surface.

20·38 The tentative design curves presented by Steele do not differentiate between thicknesses required for rigid and for flexible pavements. The modification of the design curves in the light of experience would almost certainly lead to separate curves for the two types of construction.

20·39 The group index method has not been widely used in this country up to the present time. It has been applied to the results of investigation of the causes of road failures made by the Road Research Laboratory and the actual thickness of construction is plotted against the design thickness for these results in Fig. 20·6. This figure therefore gives some indication of the reliability of the method although it should be remembered that the requirements as to

*The American No. 200 sieve is approximately equivalent to the No. 200 sieve specified in B.S. 410.

†The standard A.A.S.H.O. compaction test is equivalent to the compaction test given in B.S. 1377 (see Chapter 9).

Curve A. Thickness of sub-base required

Curve B. Total thickness of surface, base and sub-base – Light traffic

Curve C. Total thickness of surface, base and sub-base – Medium traffic

Curve D. Total thickness of surface, base and sub-base – Heavy traffic

Curve E. Thickness of additional base which may be substituted for sub-base of Curve A

Light traffic = < 50 commercial vehicles/day

Medium traffic = 50–300 commercial vehicles/day

Heavy traffic = > 300 commercial vehicles/day

FIG. 20·4 TENTATIVE DESIGN CURVES, U.S. HIGHWAY ENGINEERS GROUP INDEX METHOD

Probably the base an A-1 material and the sub-base an A-2 material

Surface

Base

Sub-base

19"

10"

or

Surface

Base

14"

Subgrade

EXAMPLE 1. Clay subgrade – A-6 Group index = 12
Medium traffic

EXAMPLE 2. Excellent gravel subgrade — A-1-a
Group index = 0 Light traffic

FIG. 20·5 EXAMPLES OF DESIGN BY GROUP INDEX METHOD

compaction and drainage given above were not necessarily fulfilled at all the sites investigated. It will be seen that several of the points in Fig. 20·6 are on the wrong side of the 45° line although failure is predicted at all the sites where definite failure actually occurred. Thus the method appears to be of value only in providing a very approximate estimation of the thickness of construction required and this is perhaps all that can be expected from such a simple method.

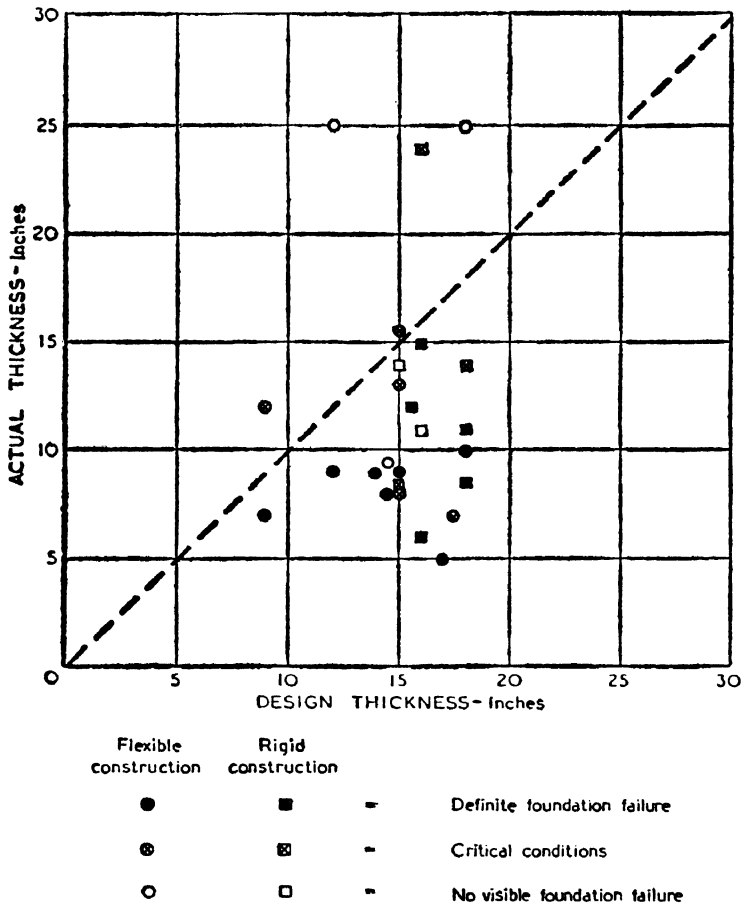



FIG. 20·6 EXPERIMENTAL EVIDENCE OF THE VALIDITY OF THE GROUP INDEX METHOD OF PAVEMENT DESIGN

Actual thickness of construction plotted against design thickness for the results of various investigations made by the Road Research Laboratory of road failures

Other Methods of Group A

20·40 Any simple set of rules for placing greater thicknesses of construction on " soft " ground than on " hard " gravel, such as was in use long before the general application of the principles of soil mechanics, may be regarded as a method of design to be classified in this group. A more elaborate method for airfields is outlined below.

20·41 CIVIL AERONAUTICS ADMINISTRATION, U.S.A.[7] METHOD. The subgrade soil is located in a classification table somewhat similar to the U.S. Highway Engineers' table, though more complicated. The table allots the soil a strength symbol varying according to the likelihood of severe frosts and the site drainage conditions. Empirical curves are given which relate the thickness of sub-base to wheel load for each strength symbol. Different sets of curves are provided for concrete pavements and for two types of flexible pavement. Similar sets of curves for the thicknesses of base and surfacing, varying with wheel load but not with soil type, are also provided. The design method includes various specifications with regard to the compaction of subgrade and sub-base, grading of sub-base and base, reinforcement in concrete pavement, etc., and stress is laid on the need for an experienced engineer in designing runway thicknesses. The design curves for this method are only applicable to aircraft wheel loads.

DESIGN METHODS OF GROUP B

EMPIRICAL METHODS USING A SOIL STRENGTH TEST

Introduction

20·42 There are two main difficulties in devising a sound theoretical method of pavement design. First, no existing analysis of the distribution of stress and strain through the layers of the pavement and subgrade has yet been proved sufficiently accurate. It is not even known whether it is necessary to take into account non-elastic behaviour of the materials. Secondly, no method has yet been devised to measure the strength properties of the materials for use in such a theory of stress distribution in order to take account of such factors as load repetition. Methods of pavement design in this group by-pass these two difficulties by making an entirely empirical approach. They select an arbitrary strength test which is considered to stress the subgrade in a similar manner to an actual wheel load. The thickness of pavement is then determined on the basis of experience of the thickness required on top of subgrades of similar strength in the past.

20·43 The first strength test which comes to mind as suitable for this type of method is a bearing test using plates of about the same area as the tyre contact area. Many methods in the group use such a test. However, the plate-bearing test necessitates bulky field equipment to supply dead load reaction and a lengthy test procedure. Many recent methods, therefore, employ small-scale bearing tests or penetration tests and their advocates consider inaccuracies due to the scale effect are well off-set by the ease of manipulation of the apparatus and the speed of operation. The most important method employing a small-scale bearing test, and in fact the most important method of the whole group, is the California bearing ratio method.

The California Bearing Ratio Method

20·44 This method was originally devised by O. J. Porter[8], then of the California State Highway Department, but it has since been developed and modified by other authorities in the U.S.A., notably the U.S. Corps of Engineers. Briefly, the test consists of causing a cylindrical plunger, 3 sq.in. in end cross-section, to penetrate a sample of soil at $\frac{1}{20}$ in./min. and measuring the load required to cause a penetration of 0·1 in. or 0·2 in. This load is expressed as a percentage of a standard load and is known as the California bearing ratio, usually abbreviated to C.B.R. Full details of the test procedure are given in Chapter 19.

20·45 The standard U.S. Corps of Engineers procedure requires the sample to be tested after soaking for 4 days to simulate the worst possible subgrade conditions.

20·46 The California State Highway Department carried out this test over a number of years on the materials comprising the subgrade, sub-base and base of roads that had and had not failed. This experience led to the conclusion

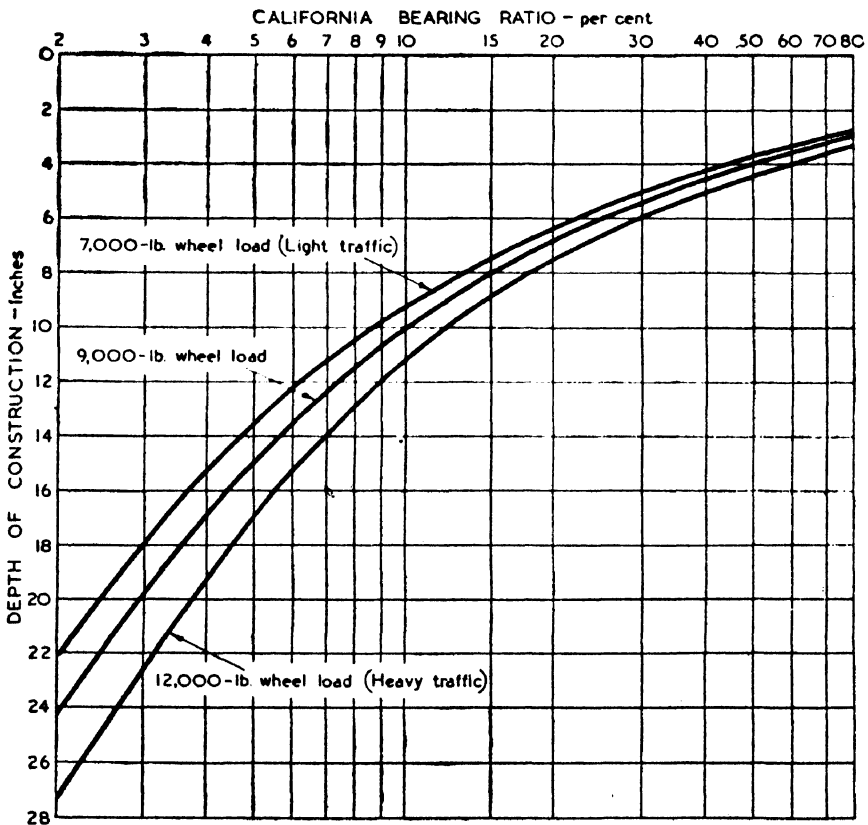



FIG. 20·7 DESIGN CURVES FOR ROADS, CALIFORNIA BEARING RATIO METHOD

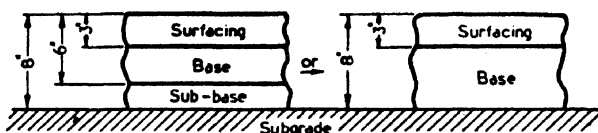
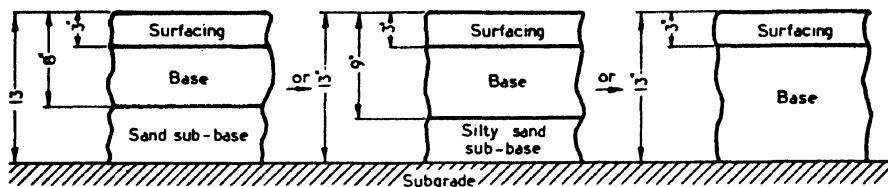
that a material of a certain C.B.R. required a certain minimum thickness of construction above it. These minimum thicknesses are those for the 7,000-lb. wheel load given in Fig. 20·7. The curve for a 12,000-lb. wheel load was obtained from subsequent experience by the California State Highway Department and the curve for 9,000 lb. was obtained by interpolation between the other two curves. Fig. 20·8 shows how the curves may be used to design the thickness of sub-base, base and surfacing. When more than one foundation material is available it is easy to compare the costs of equivalent forms of construction.

EXAMPLE 1. 12,000-lb. wheel load
Sand subgrade, C.B.R. = 20%
Poorly-graded gravel sub-base, C.B.R. = 35%
Well-graded gravel base, C.B.R. > 80%

EXAMPLE 2. 12,000-lb. wheel load
Sandy clay subgrade, C.B.R. = 8%
Sand sub-base C.B.R. = 20% or silty sand sub-base, C.B.R. = 15%
Well-graded gravel base, C.B.R > 80%

FIG. 20·8 EXAMPLES OF DESIGN BY CALIFORNIA BEARING RATIO METHOD

20·47 The design curves have been extrapolated for aircraft wheel loads and a limited number of full-scale repeated-loading tests on trial runways have been conducted in the U.S.A. to check the validity of these extrapolations up to 60,000 lb. The design curves for runways given by the originator of the C.B.R. method[9] differ slightly from those employed by the U.S. Corps of Engineers[10], but the latter are more generally accepted and are given in Fig. 20·9. Further work is being done in the U.S.A. to check the validity of the extrapolation up to 150,000-lb. wheel loads. Fig. 4·4 gives an approximate correlation between the C.B.R. value and various soil classification systems and with Westergaard's modulus of subgrade reaction (see para. 20·93.)

20·48 The C.B.R. method was devised for the design of flexible pavements and it is for this purpose that it is generally used. However, the method is sometimes used to design the total thickness of concrete pavement and granular base, in which case some slight reduction can probably be made in the total thickness given by the curves, owing to the greater ability of concrete pavements

to spread the load as compared with flexible pavements. The C.B.R. method is unsuitable for designing the proportion of thickness of concrete to the total thickness of construction. (See para. 20·81—20·101 for methods of designing thicknesses of concrete only.)

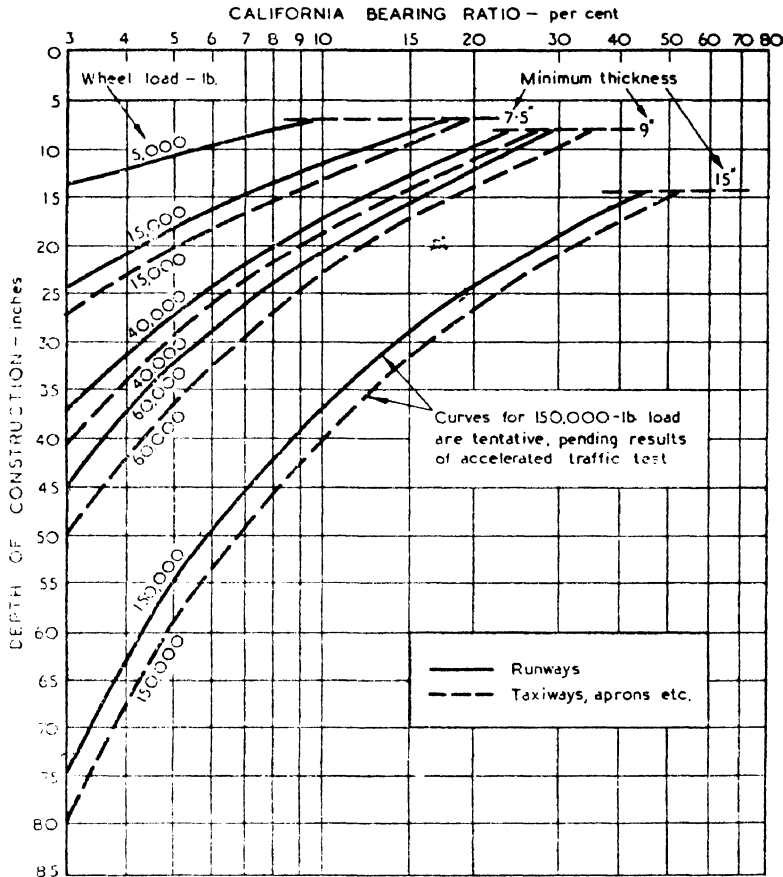



FIG. 20·9 DESIGN CURVES FOR RUNWAYS, CALIFORNIA BEARING RATIO METHOD

(U.S. Corps of Engineers)

20·49 The C.B.R. method using unsoaked specimens has been applied to the results of investigations made by the Road Research Laboratory of the failures of flexible roads, and the actual thickness of construction is plotted against the design thickness for these results in Fig. 20·10. They show that the C.B.R. method provides a fairly accurate estimate of the thickness of construction required, in that failure was always predicted by the method where it actually has occurred and never where it had not occurred. Furthermore, the points representing critical conditions tend to lie close to the lower side of the 45° line showing that the actual thickness was only a little less than the design thickness.

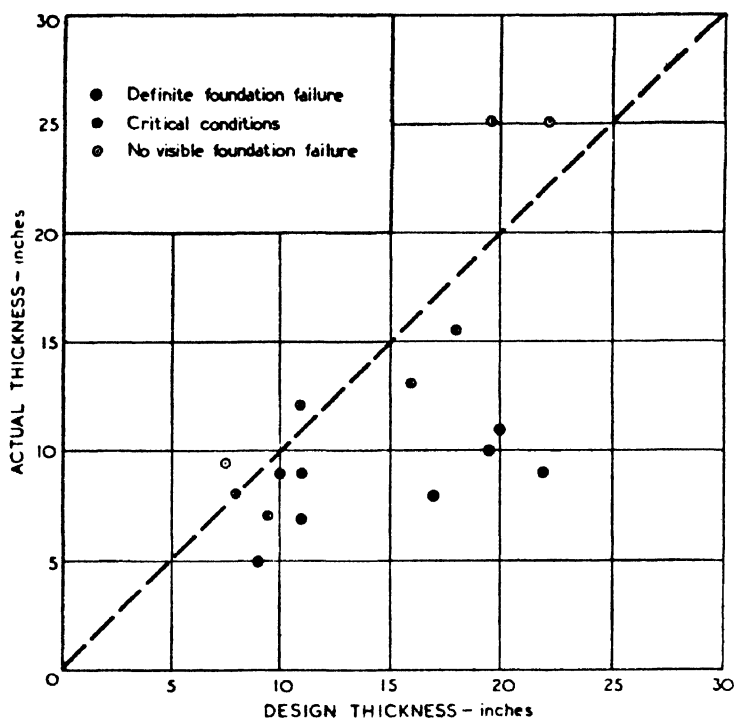



FIG. 20·10 EXPERIMENTAL EVIDENCE OF THE VALIDITY OF THE CALIFORNIA BEARING RATIO METHOD OF PAVEMENT DESIGN

Actual thickness of construction plotted against design thickness for the results of various investigations made by the Road Research Laboratory of failures of flexible roads

20·50 Both the above results for roads and the accelerated traffic test results obtained in the U.S.A. for airfields show good agreement between the required thickness of construction determined experimentally and that calculated from the design curves, provided that the C.B.R. value is determined *in situ* or on unsoaked, undisturbed samples of the subgrade after the road or test section had failed. When designing new roads or runways, the chief difficulty with the C.B.R. method is to decide under what conditions of moisture content and dry density to test the subgrade in order to allow for changes subsequent to construction, particularly in moisture content. The precaution generally followed in the U.S.A. of soaking specimens before making the penetration test is probably too severe in many cases. For example, an extensive investigation of Canadian airfields[11] made in order to develop a new method of design based on plate-bearing tests (see para. 20·57) included correlation of other strength results with C.B.R. data and showed that, whereas the C.B.R. values on soaked specimens gave thicknesses of construction considerably in excess of those found satisfactory in practice, the C.B.R. values determined *in situ* (i.e. un-

soaked) gave thicknesses in some agreement with the thicknesses found to be satisfactory. A further objection to the soaking procedure is that it produces a specimen with more sudden changes in density and moisture content within a small volume, owing to swelling and softening of the top ½ in. of soil, than are likely to occur in practice.

20·51 When using the C.B.R. method to design the thickness of construction for a new road or airfield and when it is proposed to compact the subgrade, it is probably best to remould the subgrade soil at different moisture contents and dry densities and to test the specimens in an unsoaked condition in order to plot the effect of these soil conditions on the C.B.R. value. When the road or runway is in cut and it is not proposed to compact the subgrade, C.B.R. tests are preferably made on unsoaked undisturbed specimens (especially if the soil is a clay) in order to avoid any possible loss in strength due to the destruction of natural structure within the soil by remoulding. In these circumstances it is not so easy to study the effect of changes in moisture content on the strength, but such a study may be possible if a number of undisturbed specimens are allowed slowly to become wet or dry under different humidity conditions. The C.B.R. value for economical design is then that corresponding to the moisture content likely to be the equilibrium moisture content (see Chapter 16), as shown by a study of moisture content/depth profiles for the natural soil. The estimation of the equilibrium moisture content will be facilitated by an increase in the knowledge of the factors governing moisture movements in soils.

20·52 The present position in the U.S.A. with regard to the C.B.R. method of design is reviewed in detail in a recent symposium on the subject[12].

20·53 An interesting modification of the C.B.R. method of design for roads has been suggested in Australia[13]. A standard wheel load of 5,000 lb. is considered and tentative design curves are given for different numbers of thousand of repetitions of this load. This number of repetitions is obtained from surveys of traffic intensity, allowing for anticipated traffic growth modified by factors for the desired life of the pavement, the pavement width and the frequency of soaking of the subgrade determined from climatic records. If higher wheel loads than 5,000 lb. are anticipated, the number of repetitions is increased. For example, one 9,000-lb. wheel load is considered as equivalent to 16 repetitions of the standard 5,000-lb. load.

Other Design Methods of Group B

20·54 Four other design methods in this group are mentioned in the following three sub-paragraphs. A simple empirical table of thicknesses for concrete construction, which might be included as a further method, is given on p.426.

20·55 North Dakota Cone method. A method similar to the C.B.R. has been developed by the North Dakota State Highway Department[14]. For the test associated with this method a cone penetration apparatus is used and the procedure is described in Chapter 19. The design method was developed in a similar manner to the C.B.R. method, the test being made on subgrades under roads that had failed and those that had not failed. One curve was found to

separate failures from non-failures on a graph of subgrade cone-bearing value against thickness of construction. The equation of this curve is:—

$$H = \frac{65 \cdot 7}{b^{0 \cdot 388}} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2)$$

where H = Thickness of construction (in.)

b = Cone-bearing value (lb./sq.in.)

This formula was considered to apply to an average heavy-traffic wheel load of about 5,000 lb. with a contact pressure of 70 lb./sq.in. and the method includes means of converting the thickness of construction given by the formula to a thickness for other wheel loads and contact pressures. This method has not been nearly as widely used as the C.B.R. method and is unlikely to be as reliable.

20·56 U.S. NAVY AND FLEXIBLE PAVEMENT COMMITTEE METHODS. Both the U.S. Navy Department, Bureau of Yards and Docks[15], and the U.S. Highway Research Board Committee on Flexible Pavement Design[16] recommend methods employing the plate-bearing test. Trial sections of pavement are constructed of approximately the right thickness and the results of plate-bearing tests on the top of these trial sections are used to obtain a more accurate estimate of the thickness of construction required. A similar technique has been used in this country to determine the load-carrying capacity of concrete runways, tests being made on the corners of trial slabs and continued until the slab breaks under the load.

20·57 CANADIAN DEPARTMENT OF TRANSPORT METHOD. The Canadian Department of Transport[11] recently made an extensive investigation of the stability of existing airfields in Canada using repeated plate-bearing tests correlated with *in situ* C.B.R. tests, North Dakota cone tests, Housel's penetrometer tests and triaxial compression tests. The method of pavement design evolved from the results of the plate-bearing tests is too complex to be described here but is based on the load on a plate of 30-in. diameter, after 10 repetitions of load, required to cause 0·5-in. deformation for tests both on the finished surface and on the subgrade. Using the correlation obtained between the *in situ* C.B.R. and the plate-bearing test value for the bearing capacity of the subgrade, the design curves of this method are in moderate agreement with the C.B.R. design curves.

DESIGN METHODS OF GROUP C

METHODS BASED PARTLY ON THEORY AND PARTLY ON EXPERIENCE

Introduction

20·58 To overcome the difficulty of analysing the true stress/strain characteristics of the soil and the true distribution of stress in the layers under a wheel load, methods in this group make certain assumptions and neglect some factors, thus producing a simplified theory which can be easily handled. For example, many early methods of pavement design assumed the wheel load to be spread at a fixed angle, say 45°, through base and sub-base and to be uniformly dis-

tributed over the area of subgrade thus projected. The pressure on the subgrade calculated in this way was then compared with the bearing capacity of the soil. Such assumptions are not now generally considered valid, nor is it possible to obtain one figure for the bearing capacity of a subgrade as this quantity varies with bearing area and method of loading. It is apparent therefore that the assumptions and the neglect of factors involved in any method of this group must be proved reasonable by experience. Without such proof, the method has little value, however neat and simple the theory.

Boussinesq Elastic Theory

20·59 Many methods in this group assume the subgrade, sub-base, base and surfacing to form one semi-infinite, elastic, homogeneous, isotropic solid, and consider the wheel load to be uniformly distributed over a circular area equivalent to the contact area. The distribution of stress in such an elastic solid was first analysed mathematically by Boussinesq[17], who gave equations for the stress and displacements at any point within the solid due to a point load acting normal to the surface. The equations for the stress due to a uniformly distributed load acting over a circular area can be obtained by integrating Boussinesq's equations over the area. Fig. 22·10 shows the bulbs of uniform vertical stress produced by such a loading.

20·60 The maximum vertical stress across any horizontal plane occurs on the vertical axis. The absolute maximum shear stress at any point on a horizontal plane also occurs on the axis provided the plane is below a certain depth. This depth is 0·71 × radius of loading circle when Poisson's ratio equals a half, and for circular loading.

20·61 The equations for the vertical and horizontal stresses at any point on the vertical or z axis are:—

$$\sigma_z = p\left\{ 1 - \frac{z^3}{(a^2 + z^2)^{3/2}} \right\} \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (3)$$

$$\sigma_y = \sigma_x = \frac{p}{2}\left\{(1 + 2\mu) - \frac{2(1 + \mu)z}{(a^2 + z^2)^{1/2}} + \frac{z^3}{(a^2 + z^2)^{3/2}}\right\} \ldots \quad (4)$$

σ_z = vertical stress on the axis

$\sigma_x = \sigma_y$ = horizontal stress on the axis

p = applied pressure or intensity of loading at the surface

a = radius of applied circle of loading

z = distance of the point from the surface

μ = Poisson's ratio

It is interesting to note that both expressions are independent of the modulus of elasticity, and that the vertical stress is independent of all elastic constants. This would not be so if the material had varying elastic properties.

20·62 The vertical and horizontal stresses on the axis are major and minor principal stresses respectively. The maximum shear stress at any point is half the difference between the principal stresses, therefore:—

$$\tau_{max} = \frac{\sigma_z - \sigma_x}{2}$$

$$= p \left\{ \frac{(1 - 2\mu)}{4} + \frac{(1 + \mu)z}{2(a^2 + z^2)^{\frac{1}{2}}} - \frac{3z^3}{4(a^2 + z^2)^{\frac{3}{2}}} \right\} \quad \ldots \quad \ldots \quad (5)$$

where τ_{max} = maximum shear stress at a point on the axis.

20·63 τ_{max} is also the absolute maximum shear stress at any point on a horizontal plane provided the plane is at a depth more than 0·71a, as has already been mentioned.

20·64 Equations (3), (4) and (5) are plotted in Figs. 20·11, 20·12 and 20·13. Figs. 20·12 and 20·13 assume Poisson's ratio to be a half; no such assumption is necessary for Fig. 20·11.

20·65 The vertical elastic displacement at the surface under the centre of the applied loading is given by:—

$$\Delta = \frac{2pa}{E} (1 - \mu^2) \quad \ldots \quad \ldots \quad \ldots \quad (6)$$

where E = modulus of elasticity of the solid

and Δ = displacement.

When $\mu = \frac{1}{2}$, equation (6) reduces to:—

$$\Delta = \frac{1{\cdot}5\ pa}{E} \quad \ldots \quad \ldots \quad \ldots \quad (7)$$

Equation (7) is used as the basis of those methods in Group C which design to a limiting deformation of the pavement.

20·66 If the load is applied by means of a rigid circular plate the loading is not uniformly distributed and the theoretical displacement of the plate, assuming $\mu = \frac{1}{2}$, is given by:—

$$\Delta = \frac{1{\cdot}18\ pa}{E} \quad \ldots \quad \ldots \quad \ldots \quad (8)$$

20·67 It must be remembered that the use of equations (3) to (8) in soil mechanics produces, at best, only an approximation to the true stresses and deformations, for the following reasons:—

(1) Soil only behaves elastically to a limited extent.

(2) It is not necessarily correct to assume that the applied loading is uniformly distributed.

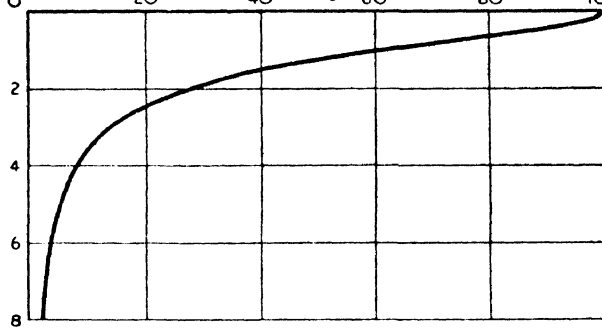



FIG. 20·11 VERTICAL STRESS ON THE AXIS WITHIN A SEMI-INFINITE ELASTIC MEDIUM DUE TO A CIRCULAR UNIFORM LOADING

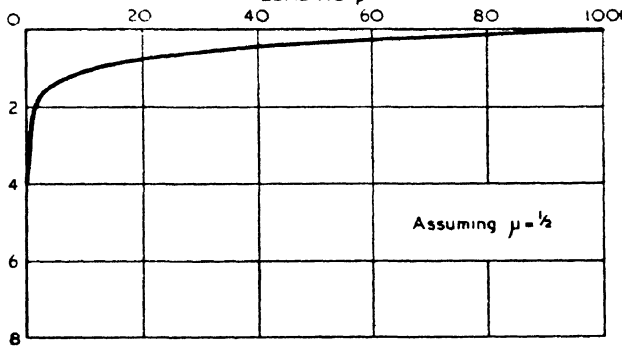



FIG. 20·12 HORIZONTAL STRESS ON THE AXIS WITHIN A SEMI-INFINITE ELASTIC MEDIUM DUE TO A CIRCULAR UNIFORM LOADING

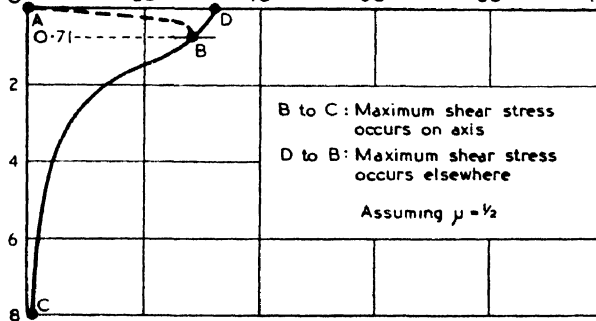



FIG. 20·13 MAXIMUM SHEAR STRESS WITHIN A SEMI-INFINITE ELASTIC MEDIUM DUE TO A CIRCULAR UNIFORM LOADING

(3) The equations are only correct provided the modu us of elasticity is constant throughout. The approximate modulus of elasticity for any given soil varies with moisture content, dry density and stress conditions. The modulus varies even more through the layers of sub-base, base and surfacing. The approximate modulus for soil can be anything from under 1,000 to 10,000 lb./sq.in.; that for flexible bases anything from 10,000 to 100,000 lb./sq.in.; and that for concrete from 2,000,000 to 6,000,000 lb./sq.in.

Shear Strength Method

20·68 In a paper presented to the Institution of Civil Engineers in 1944[18], a design method was proposed for pavements on clay subgrades, which can be classified as typical of this group of methods and which makes use of the Boussinesq theory of stress distribution.

20·69 The method assumes the pavement and subgrade to form one homogeneous elastic solid. Thus, although the method was originally suggested for rigid pavements, it is more logically applied to flexible pavements in which the modulus of elasticity of the road material and soil are more nearly equal. The method is only applicable to clay subgrades, that is, to soils with a zero or negligible angle of shearing resistance.

20·70 To quote from the paper:—" The bases of the proposed method are:—

"(1) To make a soil survey of the site; shear strength of soil samples being determined by the unconfined compression test. (The shear strength of purely cohesive soils is half the unconfined compressive strength.)

"(2) To ascertain the tyre contact areas and the load per tyre of the vehicles which the pavement will be required to carry.

"(3) To calculate the thickness of pavement required to reduce the stress in the soil below the safe value determined by the soil survey."

Details of the unconfined compression test are given in Chapter 19.

20·71 A preliminary survey of the site and a control survey are made, in which boreholes of 3-in. diameter are sunk to about 4 ft below pavement level and samples for measuring the unconfined compressive strength are taken at 6-in. intervals in those boreholes.

20·72 Three criteria of failure were considered as follows:—
The shear strength at any depth to be greater than:—

(1) the maximum vertical stresses divided by π,
(2) the mean vertical stress divided by 5·64 and multiplied by 3 (a factor of safety),
(3) the mean vertical stress divided by π.

20·73 The first and last criteria were derived from formulæ for the ultimate bearing capacity of clays used on the design of strip footings. The second criterion was derived from a formula for the ultimate bearing capacity of clays used in the design of circular footings. The criterion (possibly more logical) that the shear strength at any depth should be greater than the maximum shear stress at that depth, was not considered in the original paper.

20·74 The authors carried out compression tests on clay taken from several airfields, where the runways had and had not failed, and they found that the curve of maximum vertical stress divided by π differentiated between failure and stability. This criterion was therefore selected. Fig. 20·14 shows the results obtained on the first site investigated.

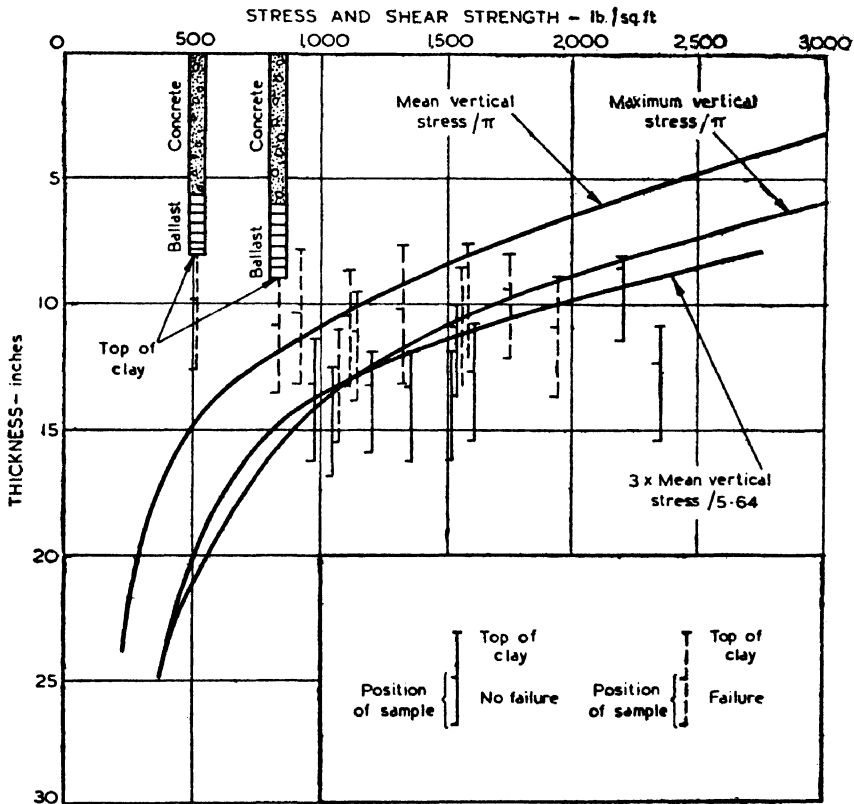
20·75 More recent investigations of road failures by the Road Research Laboratory have tended to show that the comparison of the shear strength of the subgrade soil with a Boussinesq distribution of maximum shear stress, instead of maximum vertical stress divided by π, is a more reliable procedure. Fig. 21·6 shows the results of an investigation using both criteria.

Concrete and ballast omitted in most cases for clarity
100-lb./sq. in. contact pressure
12-in. dia. equivalent circle of contact area
Wheel load = 11,300 lb.

FIG. 20·14 INVESTIGATION OF FAILURE OF RUNWAY CAUSED BY CONSTRUCTION TRAFFIC, USING ORIGINAL SHEAR STRENGTH METHOD
(Glossop and Golder)

20·76 To obtain the thickness of construction required by either criterion, the curve of stress is plotted against depth on tracing paper thus forming a mask. This mask is placed over a graph of shear strength plotted against depth to the same scale. The mask is then moved, keeping the axes of depth coincident, and the required thickness of construction is given by the minimum vertical distance between the axes of stress and shear strength for the shear strength to be greater than the stress at all depths. However, it should be noted that, if the top few inches of the clay subgrade are considerably weaker than the underlying clay, it may be more economical to cut out this weaker layer and employ the thinner construction required on the stronger clay.

20·77 The shear strength method has been applied to the results of several failure investigations made by the Road Research Laboratory, using the maximum shear stress as a criterion of failure. The design thickness is plotted against the actual thickness in Fig. 20·15. This shows that as far as these results were concerned, the method was not quite as satisfactory as the C.B.R. method although almost all the points lay on the correct side of the 45° line. Thus the shear strength method can be used with some confidence to design roads on clay subgrades, but it is better to regard the theory behind the method as a convenient way of obtaining empirical design curves rather than as a sound theory in itself. There are two main sources of inaccuracy in the theory. Firstly, it is assumed that the pavement and subgrade have the same modulus of elasticity: since the pavement is likely to be more rigid than the subgrade, this leads to an over-estimation of the stress in the subgrade. Secondly, the ultimate strength of the soil for a single application of load is used whereas it would be more logical to define the strength as the stress which produces little permanent deformation even when applied repeatedly. This leads to an over-estimation of the strength. These inaccuracies act in opposite directions and it is likely that they approximately cancel out for the range of wheel loads and soil strengths for which the method has been found satisfactory. Outside this range, i.e., for unusually weak soils or for exceptionally heavy wheel loads such as those considered when designing Class I airports, the first inaccuracy is probably much greater than the second and the method is probably not satisfactory.

Design of Rigid Pavements originating from Westergaard's Analysis

20·78 The development of methods for the design of the thickness of concrete or rigid pavements has been largely separate from that of flexible pavements. The complete design of concrete pavements also includes such matters as joint spacing, slab width, reinforcement, etc., but these are not considered here. In the design of the thickness of concrete pavements originating with Westergaard's analysis, consideration has been mainly given to the stresses within the concrete, and the subgrade has only been considered in so far as it affects these stresses. The work of Westergaard and modifications of his formulæ are best classified in this third group of design methods but they do not fit into the group as well as, say, the shear strength method.

20·79 These formulæ provide a method of calculating the stresses within a given thickness of slab. Sufficient reinforcement is not usually incorporated into concrete road or runway slabs to provide any appreciable increase in

strength, at least not theoretically, and the function of such reinforcement is considered only to control such cracking of the concrete as may occur.

20·80 The structural value of a granular base between a concrete pavement and a weak subgrade is still a matter of controversy. However, methods for designing flexible pavements such as the C.B.R. or shear strength methods are sometimes used to obtain an overall thickness of concrete and granular base. Such overall thicknesses may be conservative and a reduction of as much

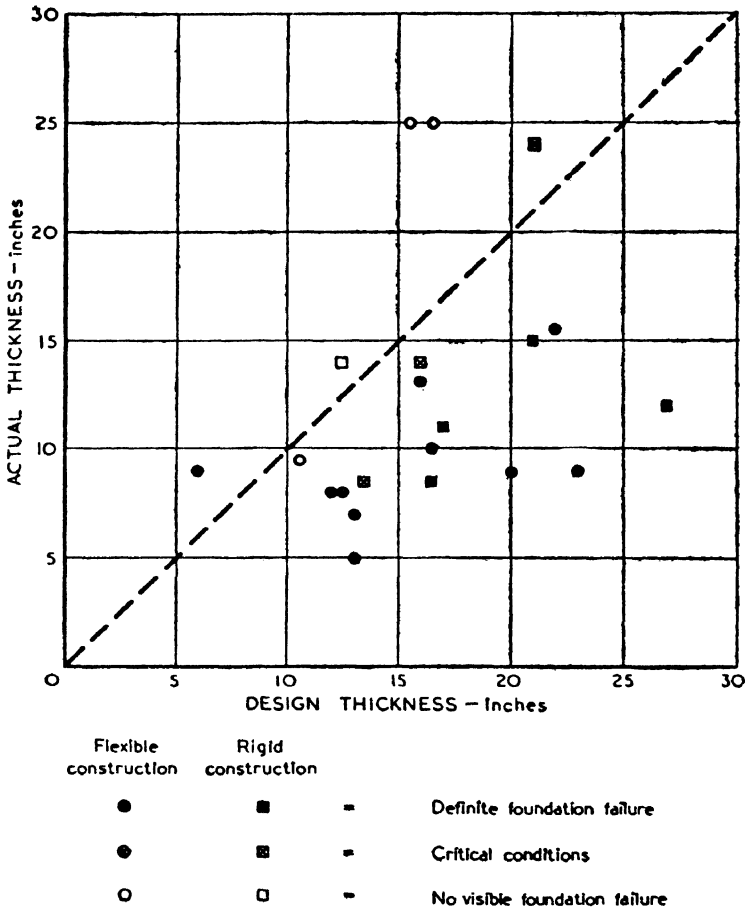



FIG. 20·15 EXPERIMENTAL EVIDENCE OF THE VALIDITY OF THE SHEAR STRENGTH METHOD OF PAVEMENT DESIGN

Actual thickness of construction plotted against design thickness for the results of various investigations made by the Road Research Laboratory of road failures

as 20 per cent is perhaps reasonable to allow for the greater rigidity of concrete as compared with wholly flexible construction. However, it should be remembered that once unreinforced concrete becomes extensively cracked it becomes, virtually, a flexible type of construction.

20·81 WESTERGAARD'S FORMULAE FOR THE CALCULATION OF THE STRESSES DUE TO WHEEL LOADS. Westergaard[19] made a theoretical analysis of the stresses in a concrete slab due to wheel loads, based on the following assumptions:—

(1) That the concrete slab is a homogeneous, isotropic, elastic solid.
(2) That the reaction of the subgrade is vertical and proportional to the deflection of the slab. Thus the subgrade is assumed to be elastic, not in all directions, but in the vertical direction only.
(3) That the wheel load is uniformly distributed over a circular contact area.

20·82 The formulæ he obtained for the maximum tensile stress in the concrete due to a wheel load at a corner, at the edge, and at some distance from the edge of the slab, respectively, are as follows:—

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left\{\frac{12(1-\mu^2)k}{E h^3}\right\}^{0\cdot15}\left\{a\sqrt{2}\right\}^{0\cdot6}\right] \quad \ldots \ldots \quad (9)$$

$$\sigma_e = \frac{0\cdot529P}{h^2}\left[1 + 0\cdot54\,\mu\right]\left[\log_{10}\left\{\frac{E h^3}{k b^4}\right\} - 0\cdot71\right] \quad \ldots \ldots \quad (10)$$

$$\sigma_i = \frac{0\cdot275P}{h^2}(1+\mu)\log_{10}\left\{\frac{E h^3}{k b^4}\right\} \quad \ldots \ldots \quad (11)$$

where

E = Modulus of elasticity of concrete (lb./sq.in.) (3 to 6 x 10^6 lb./sq.in.)
μ = Poisson's ratio for concrete (0·1 to 0·35)
k = Modulus of subgrade reaction (lb./sq. in./in.) (For determination see below)
h = Thickness of concrete slab (in.)
P = Total load exerted by one wheel (lb.)
a = Radius of equivalent circle of contact area (in.)
b = $\sqrt{(1\cdot6a^2 + h^2)} - 0\cdot675h$ when $a < 1\cdot724\,h$
b = a when $a > 1\cdot724\,h$
σ_c = Maximum tensile stress in the concrete at the top of the slab near a corner (lb./sq. in.)
σ_e = Maximum tensile stress in the concrete at the bottom of the slab along an unbroken edge (lb./sq. in.)
σ_i = Maximum tensile stress in the concrete at the bottom of the slab at an interior location, directly under the centre of an applied load (lb./sq. in.)

20·83 The conclusions to be drawn from Westergaard's formulæ (9), (10) and (11) for the stresses due to wheel loads are:—

(1) The maximum tensile stresses are directly proportional to the applied load.
(2) A considerable reduction of maximum stress occurs if the load is spread over as large an area as possible.

(3) For most practical loading conditions the greatest stresses occur when the load is located at the edge of a slab and the next greatest when the load is at the corner.

(4) There is a marked reduction of stress with increased thickness of slab. The maximum tensile stress in a 12-in. slab, for example, is only one-quarter to one-third the stress in a 6-in. slab for the same wheel load.

(5) The effects of changes of the modulus of subgrade reaction are comparatively small. A change from 200 to 50 lb./sq. in./in. in k only increases the maximum stress by 10 per cent or less.

20·84 In considering these conclusions it must be remembered that:—

(1) The theory is based on assumptions that may not be justified in all circumstances.

(2) No account is taken of secondary stresses caused by temperature gradients or other causes, which may be much more severe than those that are due to wheel loads, especially in thick slabs.

(3) The subgrade is only considered in the effect it has on the stresses in the concrete. No evaluation is made of the stresses in the subgrade itself, and these might well exceed the strength of the soil.

20·85 However, the formulæ have been widely used, especially in the United States of America, for designing concrete roads and until recently have led to the American practice of making the slab thicker at the edges than at the centre of the road.

20·86 Formulæ (9), (10) and (11) are only correct as long as 'a', the radius of applied load, is small. The contact areas of aircraft wheel loads are much larger than those for road wheel loads. Westergaard has, therefore, analysed more accurately the stresses under a wheel load some distance from the edge of the slab, in order to apply his theory to the design of concrete runways[20]. This analysis gives the stress,

$$\sigma_1' = \sigma_1 + \frac{0{\cdot}16\,Pa^2}{h^3}(1+\mu)\left\{\frac{k}{Eh}\right\}^{\frac{1}{2}} \qquad \ldots\ldots\ldots \quad (12)$$

where σ_1 is obtained from equation (11).

20·87 Making his original assumption with regard to the subgrade support but assuming the contact area to be an ellipse, Westergaard has recently made a fresh analysis of the stresses and deflections in the concrete slabs of runways[21]. These formulæ are too complicated to include here but they give the stresses due to the elliptical or any symmetrically shaped loaded area located in the interior, at an edge, or at a joint of the slab. Allowance can also be made for partial load transfer at a joint.

20·88 MODIFICATIONS TO WESTERGAARD'S FORMULÆ FOR WHEEL LOAD STRESSES. Various experiments have proved that the actual stresses can differ considerably from the theoretical stresses calculated by Westergaard's formulæ. For example, Sparkes[2] has shown that the corner stresses can be as high as those given by the following simple cantilever formula, assuming no subgrade support at the corner at all:—

$$\sigma_c = 3P/h^2 \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

20·89 However, such high stresses are rare and the true corner stress usually lies between that given by equation (9) and that given by equation (13). The actual edge loading stresses are also higher than the theoretical stresses, and from the results of an extensive series of experiments Teller and Sutherland [23] derived the following empirical modifications to Westergaard's formulæ for corner and edge loading:—

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left\{\frac{12\,(1-\mu)k}{Eh^3}\right\}^{0\cdot3} \times \left\{a\sqrt{2}\right\}^{1\cdot2}\right] \quad \ldots \quad (14)$$

$$\sigma_e = \frac{0{\cdot}529\,P}{h^2}\left[1 + 0{\cdot}54\,\mu\right]\left[\log_{10}\left\{\frac{Eh^3}{kb^4}\right\} + \log_{10}\left\{\frac{b}{1-\mu^2}\right\} - 1{\cdot}0792\right] \ldots (15)$$

20·90 The actual interior loading stresses are usually lower than those calculated by Westergaard's formula. Westergaard[23] himself has suggested a modification based on the conception that the reactions of the subgrade are more closely concentrated round the load than are the deflections. His modified formula is as follows:—

$$\sigma_i = \frac{0{\cdot}275\,P}{h^2}\left[1 + \mu\right]\left[\log_{10}\left\{\frac{Eh^3}{kb^4}\right\} - 54{\cdot}54\,Z\left\{\frac{l}{L}\right\}^2\right] \ldots (16)$$

where L is the maximum radial distance in inches from the centre of the load within which a redistribution of subgrade reactions is made, Z is a ratio of reductions of the maximum deflections, and l is the radius of relative stiffness

$$= \left\{\frac{Eh^3}{12\,(1-\mu^2)k}\right\}^{\frac{1}{4}}$$

20·91 The experimental determination of L and Z is discussed by Teller and Sutherland. Both quantities vary with pavement and subgrade stiffness, but where conditions are not known Teller and Sutherland suggest the use of $Z = 0{\cdot}2$ and $L = 5l$ which will usually give slightly conservative stresses.

20·92 A comparison of stresses calculated from equations (9), (10), (11), (13), (14), (15) and (16) is given in Fig. 20·16.

20·93 Determination of the modulus of subgrade reaction, k. Westergaard did not lay down any standard procedure for the determination of his modulus of subgrade reaction. It is always determined by a plate-bearing test but the details of the test vary. The most usual procedure is that laid down by the U.S. Corps of Engineers who use a steel plate of 30 in. diameter. The settlement of the plate is plotted against the total load divided by the area of the plate and k is then the slope or secant modulus of this graph taken over the first $\frac{1}{20}$-in. deflection. The standard 30-in. plate is sometimes too cumbersome and requires an inconveniently large dead load reaction, so that plates of smaller diameter, 20, 18 or 12 in., are often used. The value obtained with a smaller plate must be corrected by a factor to obtain the value corresponding to a 30-in. plate.

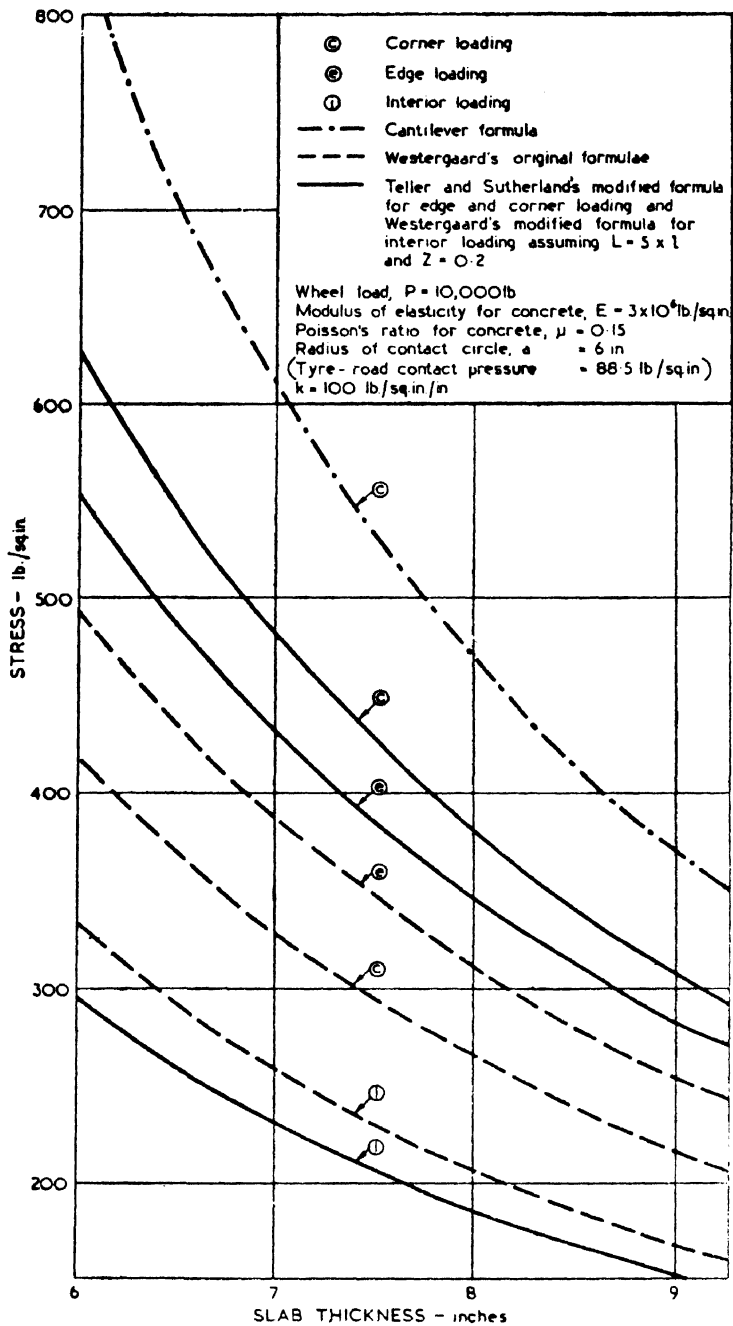



FIG. 20·16 COMPARISON OF WHEEL LOAD STRESSES IN CONCRETE SLABS OF DIFFERENT THICKNESS CALCULATED BY VARIOUS FORMULAE

20·94 Equation (8), para. 20·66, gives the vertical elastic displacement under a rigid circular plate:— $\Delta = 1{\cdot}18\ pa/E$

where p = the average vertical pressure under the plate (lb./sq.in.)
a = the radius of the plate (in.)
Δ = the displacement (in.)
and E = the modulus of elasticity of the soil (lb./sq. in.)

but, assuming the deformations of the soil under the plate are wholly elastic,

k = p/Δ where k = the modulus of subgrade reaction (lb./sq. in./in.)

therefore $k = E/1{\cdot}18a$ ... ... (17)

Thus, for settlements within the elastic range, the modulus of subgrade reaction varies inversely with the diameter of the plate. However, even a settlement of only $\frac{1}{20}$ in. can be outside the elastic range of many soils and it is often assumed for practical purposes that k does not vary appreciably with plate diameter for diameters over 30 in. There is some experimental justification for this assumption as can be seen from Stratton's[24] empirical relationship which is compared with the inverse relationship based on elastic theory in Fig. 20·17. Full details of the plate-bearing test are given in Chapter 19.

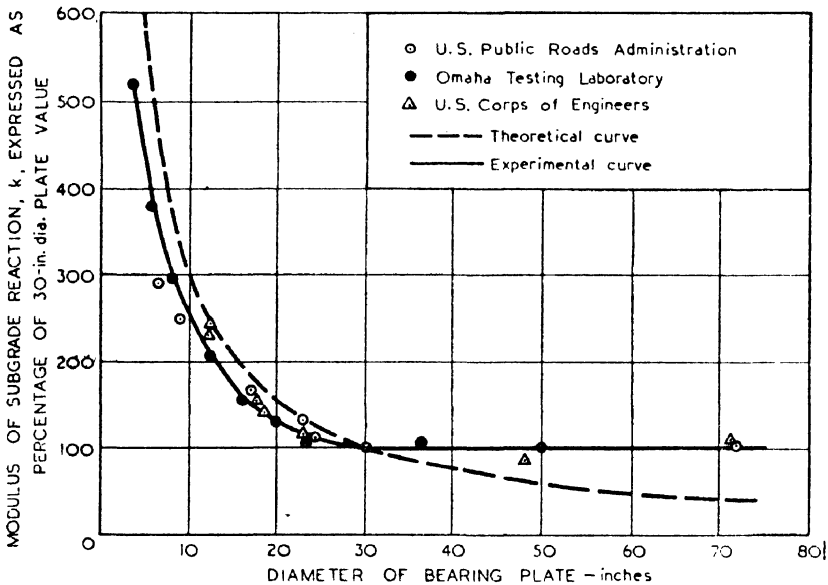



FIG. 20·17 RELATIONSHIP BETWEEN MODULUS OF SUBGRADE REACTION AND DIAMETER OF BEARING PLATE

20·95 STRESSES DUE TO TEMPERATURE GRADIENTS. When the top of a concrete slab is at a higher temperature than the bottom, as on a summer day, the slab tends to become convex upwards. When the temperature gradient is reversed, the slab tends to become concave upwards and thus the corners may be lifted clear of all subgrade support and this fact is some justification for the use of the cantilever formula (13) for corner wheel load stresses.

20·96 Westergaard[25] has analysed the problem of warping stresses due to a temperature gradient through the slab on the assumption that this gradient is a straight line. He derived the following expression for the case of a very large slab:—

$$\sigma_0 = \frac{E \, \varepsilon_t \, t_1}{2\,(1 - \mu)} \quad \ldots \quad (18)$$

where σ_0 = Tensile stress due to the temperature gradient (lb./sq.in.) (the same in all directions)

E = Modulus of elasticity of concrete (lb./sq. in.)

ε_t = Coefficient of linear thermal expansion of concrete (in./in./°F.), usually taken as 0·0000055/°F.

t_1 = Temperature difference between the top and the bottom of the slab (°F.). (Maximum t_1 in Great Britain varies between about 31°F. for a 9-in. slab and 23½°F. for a 6-in. slab.)

For a slab having $E = 5 \times 10^6$ lb./sq.in., ε_t = 0·0000055/°F., t_1 = 30°F., and μ = 0·15, then σ_0 = 485 lb./sq.in., which may well be greater than the maximum stress due to wheel loads and enough to crack the slab.

20·97 From actual temperature measurements, Thomlinson[26] showed that the temperature gradient is not in fact a straight line. He developed a theory which agrees with experimental results more closely than Westergaard's equation (18) and which gives stresses considerably lower than Westergaard's. The theory is too complex to give here but the calculated stress is based on the following factors:—

(1) Non-linear variation of temperature in a direction normal to the exposed surface,

(2) an overall change of temperature where the consequent changes in length are resisted, and

(3) a mean, uniform temperature gradient normal to the exposed surface, where the resulting warping is resisted.

20·98 Computation of the stress is easily made from tables given by Thomlinson and these should be used in preference to Westergaard's formula (18).

20·99 Stresses due to overall changes in temperature. When the overall temperature changes, stresses occur in the concrete because the expansion or contraction of the slab is resisted by the friction between the underside of the slab and the top of the subgrade or base. The magnitude of these stresses is difficult to calculate but experiments by Teller and Sutherland have shown that in general it is much less than the magnitude of the stresses due to temperature gradients. Furthermore, stresses due to overall changes in temperature are only important if they increase the total tensile stress and this is unlikely since, when the overall temperature is falling and the slab is contracting, the temperature gradients will not be at their maximum.

20·100 Stresses due to moisture changes. It is known that moisture content gradients through the depth of the concrete slab and overall moisture content changes can cause stresses in a similar manner to temperature gradients

and overall temperature changes, but little conclusive work has yet been done to determine the magnitude of these stresses. An increase in temperature is often accompanied by a decrease in moisture content, but the former causes expansion whereas the latter causes contraction. Thus the two effects tend to cancel one another and the neglect of the effect of moisture changes is an unknown factor of safety.

20·101 EMPIRICAL SLAB THICKNESS TABLES. The difficulty of calculating the magnitude of secondary stresses and the probability that they are of the same order as the stress due to wheel loads, led Markwick[27] to suggest the use of empirical thicknesses, quoting Table 20·2 as an example.

TABLE 20·2

THICKNESS OF CONSTRUCTION FOR CONCRETE ROADS (MARKWICK)

Modulus of subgrade reaction (lb./sq. in./in.)	Thickness of concrete slab (in.)			Thickness of base, roads of all types (in.)
	Trunk road	Secondary road	Housing estate road	
100 to 200	10	8	7	6
200 to 400	8	7	6	3
400 or more	7	6	5	3

This table amounts, of course, to a simple design method which can be classified in Group B, but has been given here for comparison with the design of concrete pavements based on Westergaard's analysis.

Other Design methods of Group C

20·102 There are numerous other design methods which can be classified in this group. Some employ the Boussinesq analysis in a similar way to the shear strength method, but are applicable to all types of soil in that they require the measurement of the angle of shearing resistance as well as the apparent cohesion by means of the triaxial or other shear test. Other methods start from the Boussinesq displacement equation (6) and design to a limiting displacement, whilst others make use of the ultimate bearing capacity formulæ originally devised for the design of footings.

20·103 GOLDER'S METHOD. In a method[28] proposed in 1947 for soils that have both apparent cohesion and an angle of shearing resistance, use is made of either Terzaghi's or Prandtl's footing formulæ. (See Chapter 22). It is assumed that the contact area between tyre and road is a rectangle twice as long as it is broad and that the load is spread through the pavement at an angle of 26½° to the vertical. The pressure on the subgrade, thereby projected, is equated to the bearing capacity, obtained from Terzaghi's or Prandtl's formula, divided by a factor of safety. The values of apparent cohesion and angle of shearing resistance for substitution in Terzaghi's or Prandtl's formula are obtained by a triaxial or other shear test.

20·104 This method has not been widely used. In the discussion in the original paper presenting the method, the main criticisms were as follows:—The tensile strength of the concrete should not be neglected; a formula for cal-

culating the ultimate bearing capacity under a footing for a single application of load should not be used without modification for designing pavements for which many applications of load must be considered; the assumption of an arbitrary fixed angle of spread of the load through the pavement was unrealistic.

20·105 KANSAS HIGHWAY DEPARTMENT METHOD. The Kansas Highway Department[29] use a formula suggested by Barber which makes use of the Boussinesq displacement equation (7) and designs to a limiting deformation but makes some allowance for the stiffness of the pavement relative to the subgrade. The formula is:—

$$h = \frac{a}{(E_1/E_2)^{1/3}} \left\{ \left(\frac{p}{q_a}\right)^2 - 1 \right\}^{1/2} \quad \ldots \ldots \ldots \ldots \ldots \quad (19)$$

where h = thickness of pavement (in.)

a = radius of equivalent circle of contact area (in.)

E_1 and E_2 = stress-strain or elastic moduli of the pavement and subgrade respectively (lb./sq. in.)

p = contact pressure (lb./sq. in.)

q_a = allowable bearing pressure on subgrade (lb./sq. in.)

$= \frac{E_2 \Delta_s}{1{\cdot}5a}$ (from equation (7))

where Δ_s = allowable displacement of subgrade (in.) (usually assumed to be 0·1 in.)

E_2 is determined by triaxial tests (see Chapter 19), the average of two tests done at lateral pressures of 10 and 30 lb./sq. in. being taken.

E_1 is either assumed from experience or determined by triaxial tests if possible. (See Fig. 20·18 for average values.)

20·106 V.R. SMITH'S METHOD. The design method for flexible pavements recommended by V. R. Smith[30] compares the shear strength of the soil when saturated, as determined by triaxial tests, with a shear stress curve derived from Boussinesq. His method is thus similar to the shear strength method, but is applicable to all soils, not only to purely cohesive clays. His paper also includes a consideration of deformations, making allowance for the different moduli of elasticity of the pavement and the subgrade in a somewhat similar manner to the method of the Kansas Highway Department.

DESIGN METHODS OF GROUP D

WHOLLY THEORETICAL METHODS BASED ON A MATHEMATICAL ANALYSIS OF THE STRESSES AND STRAINS

Introduction

20·107 No method of design has yet been devised that has a sound theoretical basis throughout, so that this group of methods is more of an ideal than a reality. When a better understanding has been reached of the strength properties of soil, the distribution of stress through the pavement and all the other factors involved, new design methods should approach nearer to the ideal although they may never reach it. At the present time, the best empirical methods give more reliable results than any method in this group.

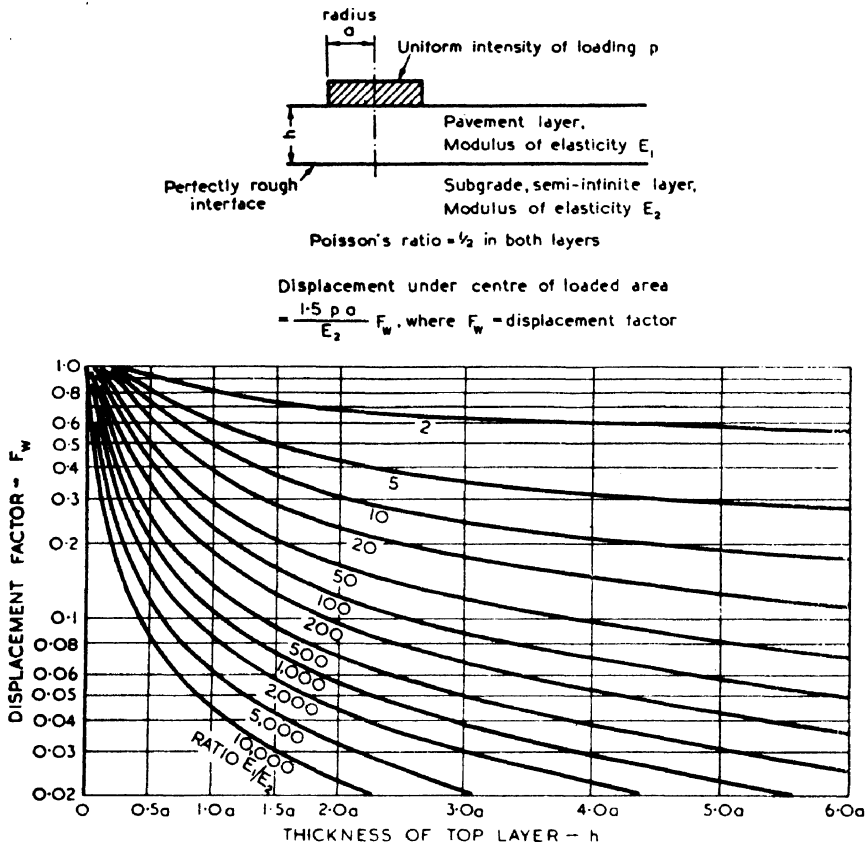



FIG. 20·18 THEORETICAL DISPLACEMENTS IN A TWO-LAYER ELASTIC SYSTEM

(Burmister)

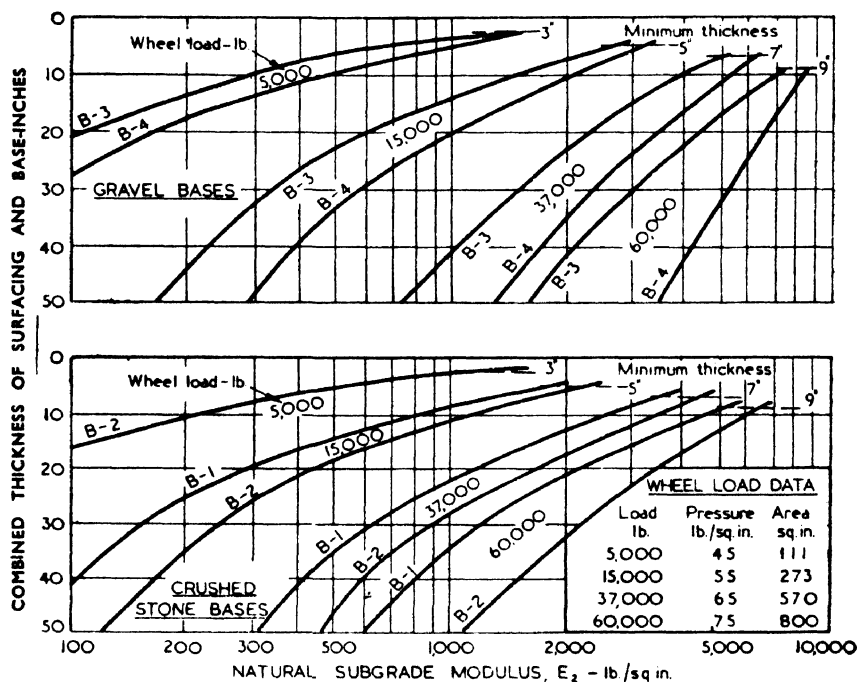
Burmister's Analysis and Design Method

20·108 THE ANALYSIS. Burmister[31] analysed the stresses and strains in a two-layered system, consisting of an elastic slab, infinite in the horizontal plane only, placed on a semi-infinite solid of lower modulus of elasticity, the system being subjected to a uniformly distributed load acting over a circular area and applied to the upper surface of the slab. The interface between the two layers is assumed to be either perfectly rough or perfectly smooth. He computed the vertical displacement at the surface under the centre of the applied load, when the interface is perfectly rough, for various ratios of the modulus of elasticity in the top to that in the bottom layer and for various ratios of the depth of the top layer to the radius of the circular area of the applied load. The results of these computations are shown graphically in Fig. 20·18. A displacement factor F_w is plotted against the thickness, h, of the top layer.

The vertical elastic displacement at the surface is then obtained from the following equation:—

$$\Delta = F_w \frac{1 \cdot 5 \, pa}{E_2} \qquad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (20)$$

where Δ = vertical displacement (in.)
p = intensity of applied load or contact pressure (lb./sq. in.)
a = radius of the circular area (in.)
E_2 = modulus of elasticity of the bottom layer (lb./sq. in.)

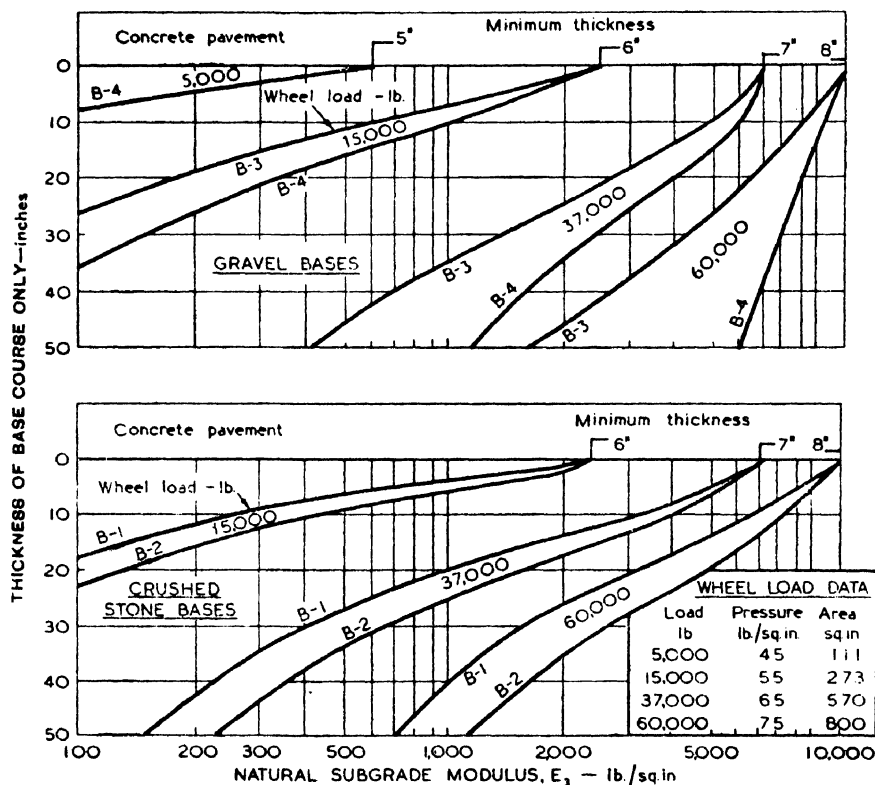
Though Burmister analysed the stresses in this system he did not compute them.

Assumptions:—Maximum limiting displacement at surface = 0.20 in.

			Modulus E_1 lb./sq. in.
Crushed stone bases:—	B-1	Best quality, maximum compaction	100,000
	B-2	Good quality and compaction	50,000
Gravel bases:—	B-3	Well graded, maximum compaction	30,000
	B-4	Run-of-bank, good compaction	15,000

FIG. 20·19 DESIGN CURVES FOR FLEXIBLE PAVEMENTS
Burmister's two-layer method

20·109 THE DESIGN METHOD. Burmister suggested a design method using the results of his analysis. For a flexible pavement the surfacing, base and sub-base are considered as the top layer and the subgrade as the bottom layer. The thickness of the top layer is selected so that the displacement under the wheel is limited to an arbitrary quantity. The value of this limiting displacement

suggested by Burmister is 0·2 in. for flexible pavements but he points out that this might have to be changed in the light of experience. The modulus of elasticity of the subgrade, E_2, is determined by a plate-bearing test using equation (17), $k = E/1{\cdot}18a$ and substituting E_2 for E, although it could be determined by triaxial tests or other means (see Chapter 19). The tentative design curves for flexible runway pavements, using 0·2 in. as the limiting deformation, are given in Fig. 20·19 together with approximate values of the modulus of elasticity, E_1 of various types of base. The extent of the experimental basis for these moduli is not given.

20·110 For rigid pavements, Burmister extended these computations to cover approximately the case of three layers—concrete slab, base and subgrade. The thickness of concrete is selected according to wheel load and experience of the thickness required to combat secondary stresses. The thickness of base

Assumptions:—Maximum limiting displacement at surface = 0.05 in.
Modulus of concrete E_1 = 3,000,000 lb./sq. in.

			Modulus E_2 lb./sq. in.
Crushed stone bases:—	B-1	Best quality, maximum compaction	100,000
	B-2	Good quality and compaction	50,000
Gravel bases:—	B-3	Well graded, maximum compaction	30,000
	B-4	Run-of-bank, good compaction	15,000

FIG. 20·20 DESIGN CURVES FOR RIGID PAVEMENTS

Burmister's three-layer method

is then selected so that the displacement under the wheel is limited to 0·05 in. (or other value if required by experience). The tentative design curves for rigid runway pavements are given in Fig. 20·20.

20·111 Burmister's method does not fully qualify for classification in Group D, for the following reasons:—

(1) The non-elastic behaviour of the materials is ignored.
(2) An arbitrary value for the limiting displacement has to be chosen. It is assumed that the pavement fails when this limiting displacement is reached, no direct account being taken of the stresses in the materials.
(3) No account is taken of traffic intensity or of deformation due to traffic consolidation and compaction.

20·112 The analysis is likely to prove very useful in the development of future Group D methods, but Burmister's method in its present form is not really practicable. The thickness of base required can be extremely sensitive to the value of limiting displacement chosen, as is shown by Fig. 20·21. Furthermore, on subgrades having the same modulus of elasticity, a greater thickness of base may be required under a concrete slab than the total thickness of construction for a flexible pavement, if Burmister's values of the limiting displacement are used. For example, consider a 37,000-lb. wheel load, a subgrade of modulus 3,000 lb./sq. in. and a well graded gravel base with maximum compaction. Referring to Figs. 20·19 and 20·20, the total thickness of flexible surfacing and base required is 16 in. whereas the thickness of concrete slab required is 7 in. and of base is 18 in., a total of 25 in. This would hardly seem to agree with experience.

20·113 EXTENSION OF THE ANALYSIS FOR THREE LAYERS. Burmister[32] has analysed the displacement under the centre of the loaded area for a three-layer system, but has not published any computations from this analysis, owing to its complicated nature. He did not analyse the stresses in the three-layer system.

20·114 PALMER AND BARBER'S APPROXIMATE DISPLACEMENT EQUATION FOR TWO-LAYER SYSTEM. In the discussion on a paper by Palmer and Barber [33], Barber gave the derivation of an approximate formula for the vertical elastic displacement at the surface of the subgrade, Δ_s, in the following manner:—

Assume the relative stiffness of pavement to subgrade

$$= \left\{\frac{E_1\,(1 - \mu_2^2)}{E_2\,(1 - \mu_1^2)}\right\}^{\frac{1}{3}} = \left(\frac{E_1}{E_2}\right)^{\frac{1}{3}} \quad \text{for } \mu_1 = \mu_2 = \tfrac{1}{2}$$

Therefore the pavement thickness, h, may be replaced by an equivalent thickness of subgrade, h_e, where:— $h_e = h\,(E_1/E_2)^{\frac{1}{3}}$.

The vertical displacement Δ_z at depth z from the surface of a semi-infinite elastic solid with uniform circular loading is given by

$$\Delta_z = \frac{1{\cdot}5\,pa^2}{E\,(a^2 + z^2)^{\frac{1}{2}}} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (21)$$

(For $z = 0$ this becomes equation (7).)

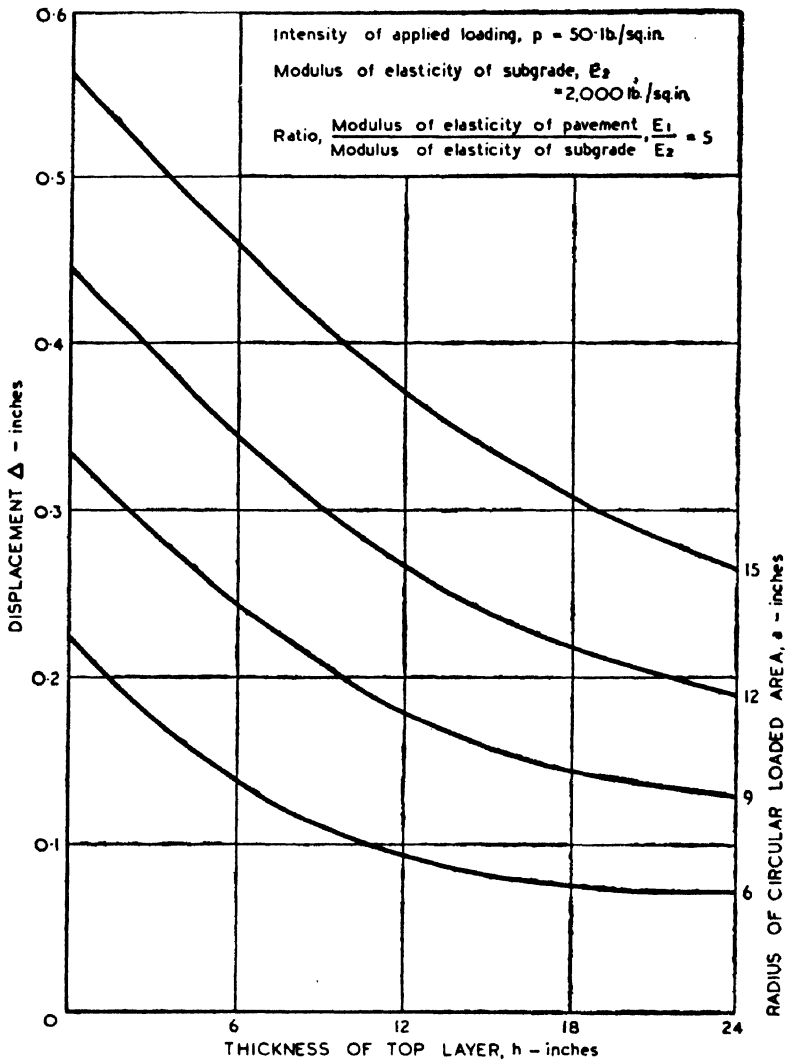



FIG. 20·21 INFLUENCE OF THICKNESS OF TOP LAYER ON DISPLACEMENT AT SURFACE OF A TWO-LAYER SYSTEM (BURMISTER)

Substituting E_2 for E and h_e for z in this equation, the vertical displacement at the surface of the subgrade, Δ_s, is given by:—

$$\Delta_s = \frac{1 \cdot 5\ pa^2}{E_2\left[a^2 + h_e^2\left(\frac{E_1}{E_2}\right)^{\frac{2}{3}}\right]^{\frac{1}{2}}} \quad \dots \dots \dots \dots \dots \dots \quad (22)$$

(Equation (19) is a rearrangement of this equation.)

In the discussion on Burmister's[31] paper, Barber carries the argument further.

The displacement within the pavement, Δ_p, is given by:—

$$\Delta_p = \frac{E_2}{E_1}\left\{\frac{1 \cdot 5\ pa}{E_2} - \Delta_s\right\} \quad \ldots \quad (23)$$

Therefore the displacement at the surface of the pavement, Δ, is given by:—

$$\Delta = \Delta_s + \Delta_p = \frac{1 \cdot 5\ pa}{E_2}\left[\frac{a}{\left[a^2 + h^2\left(\frac{E_1}{E_2}\right)^{\frac{2}{3}}\right]^{\frac{1}{2}}}\left(1 - \frac{E_2}{E_1}\right) + \frac{E_2}{E_1}\right]$$

$$= \frac{1 \cdot 5\ pa}{E_2} F'_w \quad \ldots \quad (24)$$

where F'_w is a displacement factor similar to Burmister's F_w in equation (20). A comparison between F'_w and F_w shows the following close agreement:—

TABLE 20·3

COMPARISON BETWEEN BURMISTER'S AND PALMER AND BARBER'S DISPLACEMENT FACTORS

E_1/E_2	a/h	F_w	F'_w
10,000	10	0.40	0.42
10,000	5	0.22	0.23
10,000	1	0.05	0.05
100	10	0.91	0.91
100	5	0.76	0.74
100	1	0.23	0.22
2	1	0.80	0.81

Extension of Burmister's Analysis

20·115 At the request of the Road Research Laboratory, the Mathematics Division of the National Physical Laboratory[34] extended Burmister's analysis of the stresses and strains in a two-layer system and computed the stresses for various cases. The distributions of vertical and horizontal stress on the axis in the lower layer for E_1/E_2 = 1 to 1,000 and for a/h = ½ to 2, for both a perfectly rough and a perfectly smooth interface, were computed from Burmister's analysis. The vertical and horizontal stresses on the axis in the lower layer at the interface were also computed for E_1/E_2 = 1 to 10,000 and a/h = ½ to 4.

20·116 Using the relaxation methods of Professor Southwell, the distributions of all stresses throughout the two layers were computed for a/h = 1 and E_1/E_2 = 1 to 100 for a perfectly rough interface. Poisson's ratio was assumed to be a half in both layers for all computations.

20·117 Some of the results of these computations are given in Figs. 20·22 and 20·23. Fig. 20·22 shows a considerable theoretical deviation from the Boussinesq distribution of stress, even when the pavement layer has a modulus of elasticity only ten times that of the subgrade. For example, when a subgrade of modulus 1,000 lb./sq.in. is loaded directly with 100-lb./sq. in. contact pressure on a circle of radius 6 in., the vertical stress at a depth of 6 in. on the axis is

(a) Vertical stress

(b) Shear stress

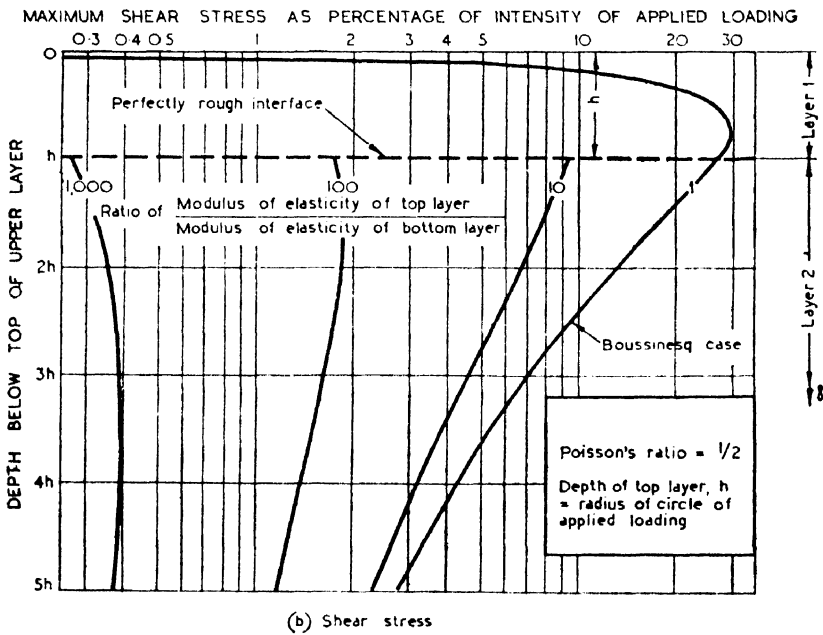
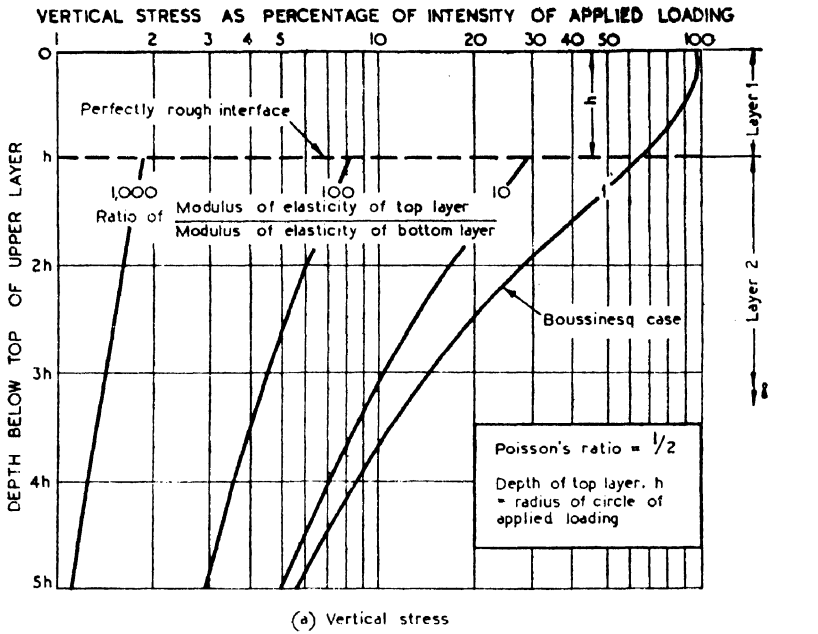



FIG. 20·22 STRESS ON THE AXIS IN THE LOWER LAYER OF A TWO-LAYER ELASTIC SYSTEM DUE TO A CIRCULAR UNIFORM LOADING

65 lb./sq. in., whereas, if this top 6 in. of subgrade is replaced by a pavement of modulus 10,000 lb./sq. in., the stress in the subgrade is reduced to 29 lb./sq.in. This reduction is further illustrated by Fig. 20·23, which shows how the bulbs of uniform vertical pressure are concentrated into the top layer for $E_1/E_2 = 10$ as compared with the more evenly spaced bulbs of the Boussinesq case. That this theoretical reduction in stress does occur in practice has yet to be verified.

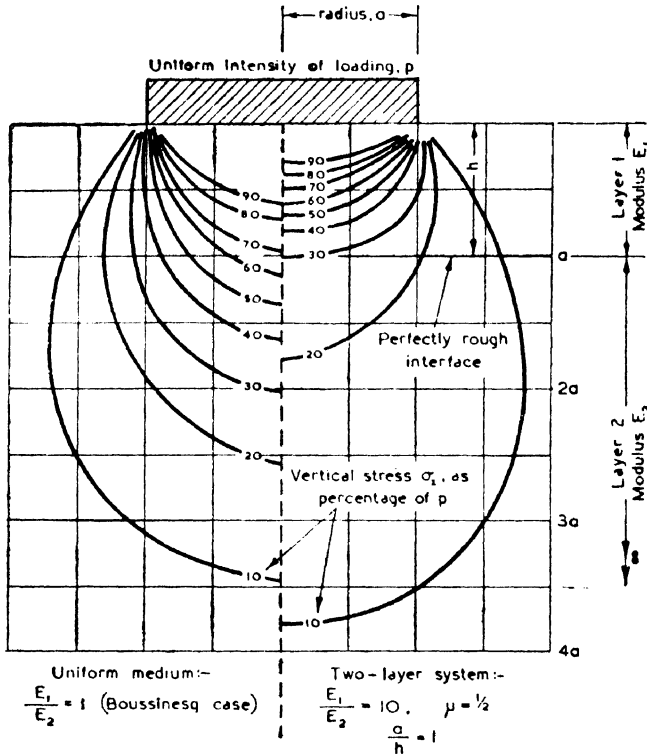



FIG. 20·23 COMPARISON BETWEEN THE VERTICAL STRESS DISTRIBUTION IN A UNIFORM MEDIUM AND IN A TWO-LAYER SYSTEM

20·118 No design method at present employs the results of these computations, though they form a very useful basis for research work in developing a satisfactory Group D method. The mathematical work is being extended to cover the case of three layers and work is in hand both in this country and in the U.S.A. to determine experimentally the distribution of stress within layered systems of road materials and soil. Even when a sound theory of distribution of stress has been developed, there is still the problem of how to measure the strength properties of the various materials for use in such a theory of stress distribution, since the measured strength must take account of such factors as repetition of load, distribution of traffic and changes in moisture content of the materials.

DISCUSSION AND CONCLUSIONS

PRESENT AND FUTURE DESIGN METHODS

20·119 Pavement design is still in a rather confused state. As yet, there are no generally accepted design methods such as are used in other older branches of engineering design. There are nearly fifty different design methods, many of which have little in common with one another, either in the assumptions they make or in the results they produce. However, it is possible to classify all design methods roughly into four groups, two of which comprise wholly empirical methods, one semi-empirical and one wholly theoretical method.

20·120 An ideal method would take account of the true strength and deformation characteristics of the materials in each layer, at all times during the life of the road. It would also take account of the true distribution of stress throughout the road and subgrade, together with factors for the anticipated traffic intensity and distribution of wheel loads across the road width. The method should then give the thickness and type of construction that will most economically ensure a desired life before surface irregularities require the application of a new surfacing. No design method can yet do this reliably.

20·121 Probably the most reliable methods at the moment are:—

(1) The C.B.R. method for flexible pavements.

(2) Westergaard's method with subsequent modifications for rigid pavements.

(3) For cohesive soils only, and where it is desired to use a simpler test than the C.B.R. or plate-bearing test, the shear strength method gives fairly reliable results for either flexible or rigid pavements. Some modifications to the original method are necessary.

20·122 Developments in the immediate future that would probably be of most use to British road engineers are:—

(1) For the design of important roads, the modifications of the C.B.R. method to suit British conditions, probably dispensing with the soaking technique and requiring the development of a reliable means of estimating the equilibrium moisture conditions of the subgrade beneath the constructed pavement.

(2) For the design of less important roads, the development of a simpler, more approximate method than the C.B.R. depending on soil classification alone, i.e. a Group A method, to suit British conditions, possibly something on the lines of the United States Highway Engineers' Group Index method.

20·123 Long-term researches in design should eventually produce a reliable Group D method. The beginnings of such a method can be seen in the elastic two-layer stress and strain analyses. Modifications to take account of the non-elastic behaviour of soil and road materials are probably necessary. Test procedure may well include repetitional triaxial testing to determine the full strength and deformation characteristics of the materials.

SUMMARY

20·124 The general factors affecting pavement design are outlined and the necessity for employing a design method is emphasized. Methods of design are classified in four groups·—

Group A, empirical methods using soil classification tests for comparing with past experience of the thickness of construction required.

Group B, empirical methods using a special soil strength test for comparing with past experience.

Group C, methods using a simplified theory of stress distribution and soil strength, supported by the justification of experience.

Group D, wholly theoretical methods using a true analysis of stress distribution and soil strength.

Each of these groups is dealt with in a separate section and at least one typical method in each group is given in some detail together with others in outline. Conclusions are drawn as to the most satisfactory design methods to use at the present time and the probable future trend of pavement design.

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CHAPTER 21

THE INVESTIGATION OF FOUNDATION FAILURES

INTRODUCTION

21·1 This chapter describes a method developed by the Road Research Laboratory for examining road foundation failures and gives examples of its application to the study of the causes of:—

(1) Cracking and disintegration of portions of a concrete road.

(2) Settlement and disintegration of a section of a bituminous road.

(3) Differential movements between slabs of a concrete road.

21·2 Such investigations of failure in the foundation of a road, help the road engineer to decide on the best methods of repair and give the research worker information of value in his studies of pavement design. In addition the investigations sometimes reveal new problems of interest both to the research worker and the maintenance engineer.

21·3 One of the main difficulties encountered in investigating causes of defects in roads is the absence generally of information relating to the conditions of the subgrade at the time when the road was constructed. This difficulty has been partly overcome by the procedure adopted by the Road Research Laboratory in which a comparison is made of the condition both of the subgrade and the construction at neighbouring failed and sound sections of road. This procedure has been found in practice to be valuable in determining the various factors responsible for the road defects.

21·4 The terms used to describe the main constructional layers of a road are the surfacing, base, sub-base and subgrade and these are shown diagrammatically in Fig. 20·1.

PROCEDURE EMPLOYED IN THE INVESTIGATION

21·5 As mentioned above, two areas are selected, one which has failed and a neighbouring one which is sound. In selecting failed areas for investigation, care has to be exercised to choose sites where little further deterioration is likely to have occurred in the subgrade subsequent to the initial failure, due to the entry of water through unsealed cracks in the road surface. Trenches about 3 ft wide are cut through the road construction to expose the subgrade at these sites. Half the width of the carriageway is examined at a time in order that the road may be kept open to traffic.

21·6 A close examination is made of the road construction at each site and measurements are made so that an accurate cross-section of the actual road construction may be drawn. In the case of concrete roads the position and

quantity of any reinforcement is noted and cores may be cut for strength tests. The following tests are usually carried out at each site.

Soil Type

21·7 Boreholes are sunk in the top 4 ft of subgrade using a post-hole auger of 4-in. diameter and representative samples (each weighing approximately 1 lb) are taken of each soil type encountered. These samples are sealed in tins and removed to the laboratory for subsequent examination at the conclusion of the field work. Particle-size analyses and index tests are carried out on selected samples, the remainder being retained for any possible future tests.

Moisture conditions

21·8 The distribution of moisture in the subgrade is found by sinking boreholes about 4 ft deep at 1- to 2-ft intervals across the road and taking samples of soil at from 1- to 6-in. intervals in each of the boreholes for the determination of moisture content. By plotting the variation in moisture content with depth, a diagram can be constructed showing the distribution of moisture in the subgrade across the road. At each site, one borehole is sunk to locate the water-table if the latter should be within 8 ft of the surface.

Dry Density Tests

21·9 Measurements are made of the dry density in the top 18 in. of subgrade using either the core-cutter method or the sand-replacement method, depending upon the nature of the soil. A British Standard compaction test is carried out on samples of the subgrade soil enabling the relative compaction of the subgrade to be found (see Chapter 9).

Shear Strength Tests

21·10 When the subgrade consists of a clay soil reasonably free from stones, the variation in the shear strength of the soil with depth is found, using the portable unconfined compression apparatus (see Chapter 19). The assumption is made that the shear strength of the soil is equal to half the compressive strength. These results are used in the shear strength method of design discussed in Chapter 20.

California Bearing Ratio Tests

21·11 When possible, a number of undisturbed samples of soil are obtained, generally from the top foot of subgrade at each site, and stored in air-tight containers. The California bearing ratio test, details of which are given in Chapter 19, is carried out in the laboratory on these specimens. In the investigations described in this chapter the original C.B.R. procedure was used in which the test was carried out on specimens at their natural moisture content and after soaking for four days. When it is not possible to obtain undisturbed samples, remoulded specimens are tested after being compacted to the dry density of the subgrade.

21·12 The results of the California bearing ratio tests are used in conjunction with the design curves given in Fig. 20·7, in the estimation of the thickness of pavement.

Plate-bearing Tests

21·13 At each site several load/deflection measurements using circular steel plates are made on the surface of the subgrade and on the sub-base if one is included in the construction. The apparatus used is shown in Plates 21·1A and B. An 18-in. diameter plate is often employed, as this size is found more convenient to use and is thought to reproduce more closely the state of stress produced by the tyre of the normal road vehicle than the 30-in. diameter plate sometimes specified. The load/deflection measurements enable values of the modulus of subgrade reaction (" k " lb./sq.in./in.) to be found.

21·14 The field work is carried out by an officer aided by two or three assistants. A mobile laboratory is stationed at the site during the investigation and, if desired, all the soil tests can be carried out in the field. The mobile laboratory used in the investigations is shown in Plate 21·1A.

Depth (in.)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
17	72	46	26
24	73	30	43
36	88	37	51

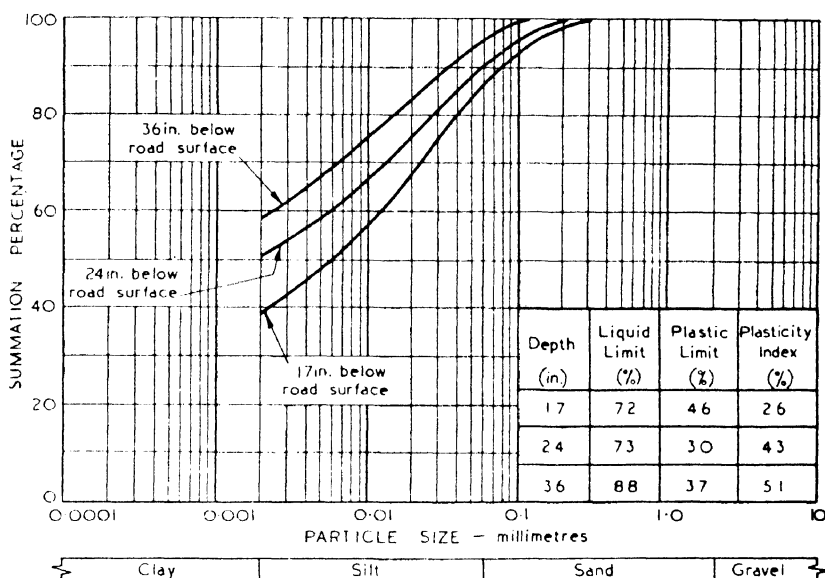



FIG. 21·1 PARTICLE-SIZE CURVES AND RESULTS OF INDEX TESTS FOR SUBGRADE SOIL BENEATH CRACKED AND DISINTEGRATED SLABS

APPLICATION OF THE TECHNIQUE

21·15 To illustrate the employment of the technique in practice, three investigations of road failures are described.

An Investigation of the Causes of Cracking and Disintegration of portions of a Concrete Road

21·16 The road at the site of the investigation was constructed in 1924 and consisted of a singly reinforced concrete carriageway 24 ft wide with no central longitudinal expansion joint. Apart from the formation of a longitudinal crack along the crown of the carriageway, the road carried normal traffic satisfactorily

until 1944 when unusually heavy traffic caused the rapid deterioration of many of the slabs (see Plate 21·2).

21·17 The concrete and subgrade were examined in accordance with the usual practice at two sites, site 1 where severe cracking had occurred and site 2 where there was only slight cracking of the slabs.

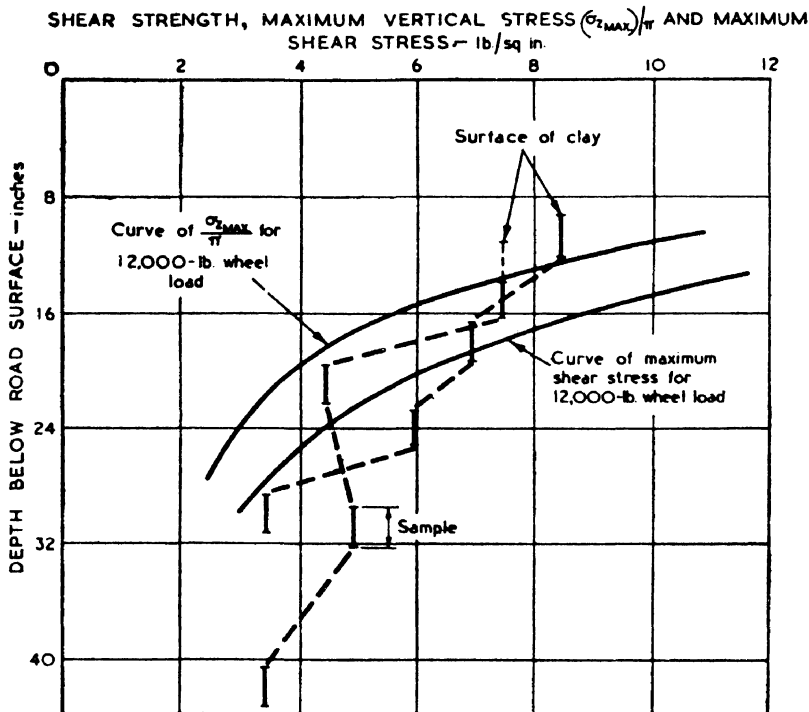



FIG. 21·2 VARIATION WITH DEPTH OF SHEAR STRENGTH OF SUBGRADE AT SITE OF SEVERE CRACKING

(Site 1)

Clay subgrade. 12,000-lb. wheel load

21·18 At site 1, there were 5-6 in. of concrete (the bottom 3 in. being badly honey-combed) and about 3 in. of sub-base, while at site 2 there were 7 in. of better quality concrete and 7 in. of sub-base. The subgrade soil was a heavy clay; the particle-size curves for samples of soil from different depths together with the results of the index tests are given in Fig. 21·1. The average moisture content of the top 2 ft of subgrade was 44 per cent at site 1 and 38 per cent at site 2, the difference being probably due to the entry of surface water through the cracks in the concrete at site 1.

21·19 The results of plate-bearing tests, in this instance using a 30-in. diameter plate, are given in Table 21·1. It will be seen that the subgrade had a much higher bearing value at site 2 than at site 1 where failure had occurred.

21·20 Figs. 21·2 and 21·3 show shear strength/depth profiles for the subgrade at the two sites, together with theoretical curves showing the variation with depth of both the maximum vertical stress/π and the maximum shear stress for a 12,000-lb. wheel load. The thickness of base and surfacing required by the original shear strength method of design* and its modification using the maximum shear stress criterion for the subgrade at the time of the investigation are given in Table 21·2, together with the actual thickness of road at the two sites. It will be seen that both methods of design indicate that the thickness of construction at site 1 was insufficient.

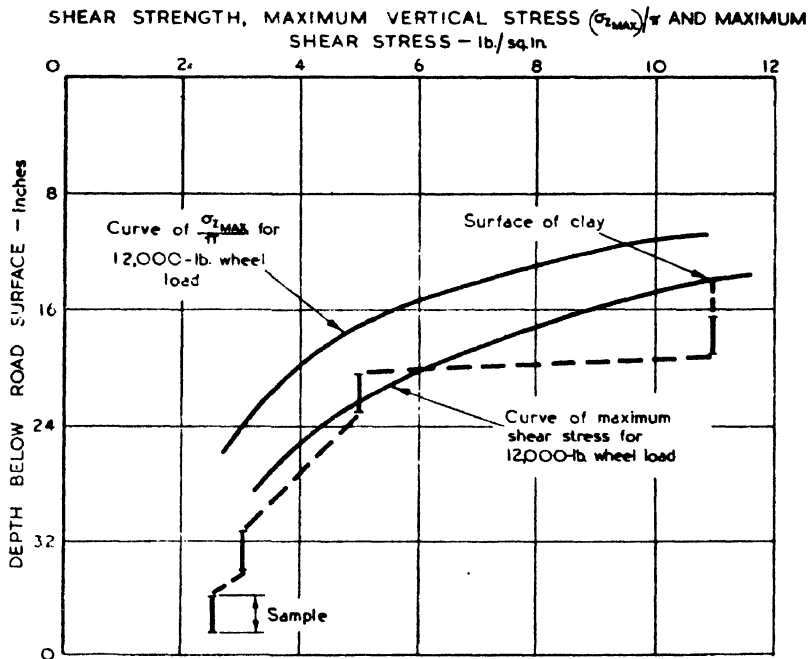



FIG. 21·3 VARIATION WITH DEPTH OF SHEAR STRENGTH OF SUBGRADE AT SITE OF SLIGHT CRACKING

(Site 2)

Clay subgrade. 12,000-lb. wheel load

21·21 The 12,000-lb. wheel load used in the calculations for heavily trafficked roads is probably very rarely encountered in practice but when used with the above design methods it has been found to give results in closer agreement with practice than the 9000-lb. wheel load which is the legal limit for normal road vehicles in this country (4 tons). It is possible that the assumption of a larger wheel load than occurs in practice may, in effect, be equivalent to taking account of such factors as impact and repetition of load.

21·22 The conclusions arrived at from the investigation were (a) that the failed areas on the road developed as a result of insufficient thickness of

*See chapter 20, reference 18.

concrete and sub-base to carry the very heavy traffic, and (b) that a total thickness of base and surfacing of 16 in. would have been adequate to prevent the occurrence of any serious defects. The continuous crack along the crown of the road would have been avoided by the use of a central longitudinal joint.

TABLE 21·1

VALUES OF "k" OBTAINED BY PLATE-BEARING TESTS FOR SITES INVESTIGATED

Location of test	Plate-bearing values " k " (lb./sq.in./in.)			
	Site 1 (extensive cracking)		Site 2 (slight cracking)	
	On sub-base over sub-grade	On subgrade (sub-base removed)	On sub-base over sub-grade	On subgrade (sub-base removed)
East half of road	101	98	179	130
West half of road	87	58	173	146
Average value	94	78	176	138

TABLE 21·2

THICKNESS OF ROAD CONSTRUCTION AT SITES INVESTIGATED WITH THEORETICAL VALUES GIVEN BY THE ORIGINAL SHEAR STRENGTH METHOD AND A MODIFICATION USING THE MAXIMUM SHEAR STRESS AS A CRITERION

Site investigated	Actual thickness of road construction (in.)	Thickness given by original shear strength method (in.)	Thickness given by modified shear strength method (in.)
Site 1 (extensive cracking)	9	12	16
Site 2 (slight cracking)	14	11	15

An Investigation of the Causes of Settlement and Disintegration on a section of a Bituminous-surfaced Road

21·23 A bituminous-surfaced road 24 ft wide had shown severe settlement accompanied in some places by disintegration especially along the crown and northern side of the road (see Plate 21·3). This section of road had been a source of trouble for a considerable period of time and approximately £8,000 had been spent on maintenance over a period of 8 years prior to the investigation.

21·24 The cross-section of the road (Fig. 21·4) showed a considerable variation in the thickness of construction from about 30 in. along the kerbs to about 10 in. along the crown. A 9-in. open-jointed drain with rubble surround was located on the northern side of the road at a depth of about 3 ft.

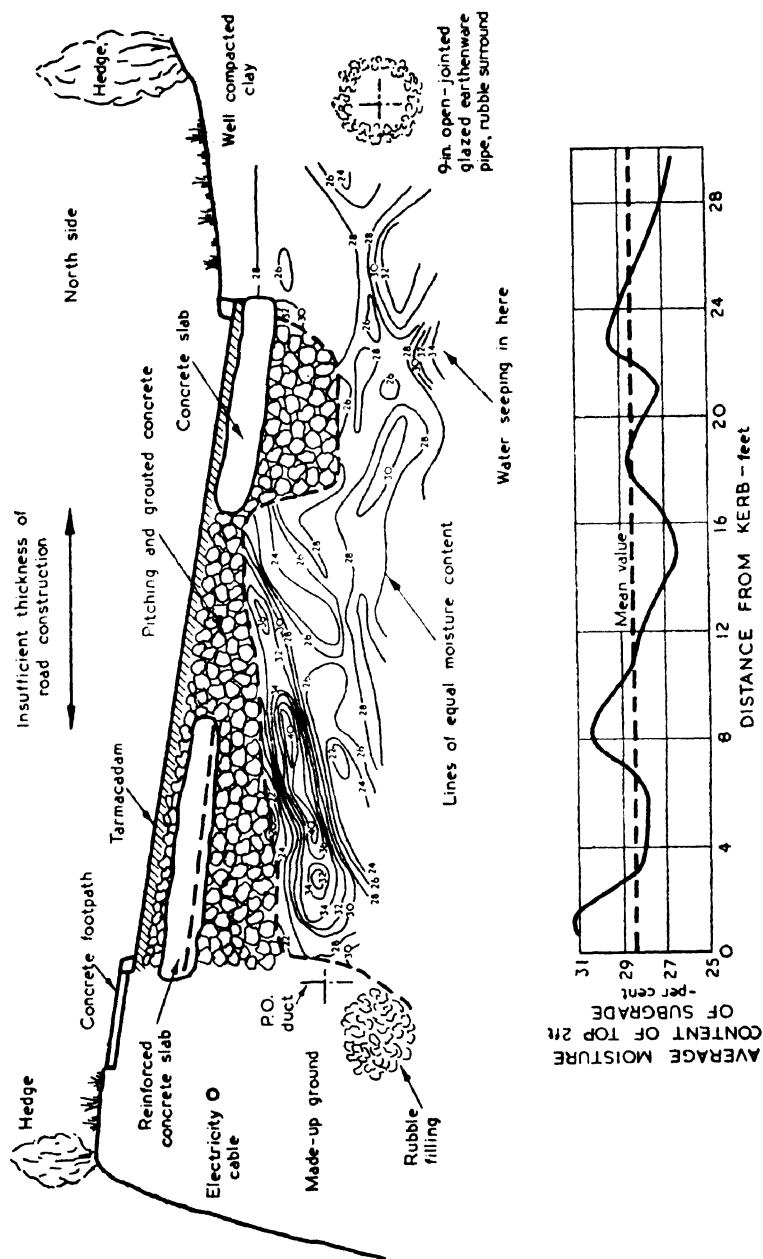



FIG. 21·4 CROSS-SECTION OF BITUMINOUS-SURFACED ROAD AT SITE INVESTIGATED
Showing construction of road and the moisture distribution in the clay subgrade

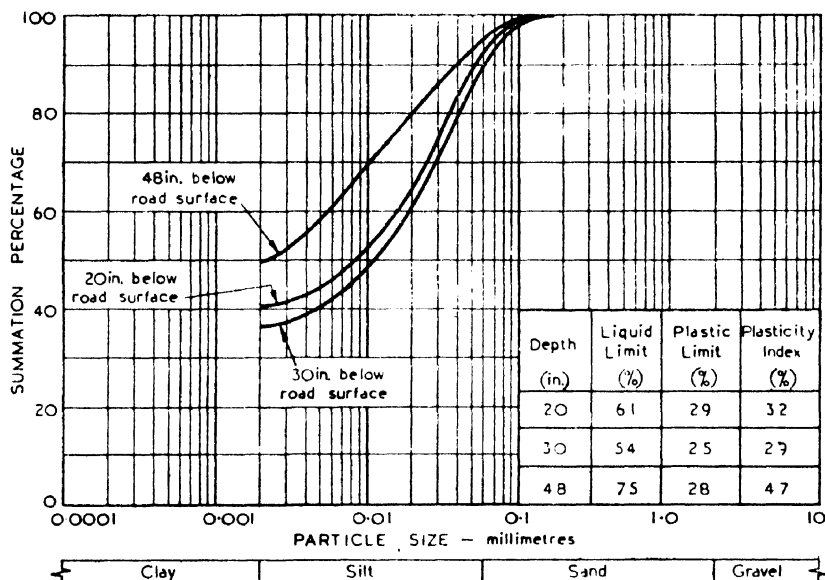



FIG. 21·5 PARTICLE-SIZE CURVES AND RESULTS OF INDEX TESTS FOR SUBGRADE SOIL

21·25 The subgrade soil consisted of a rather silty clay, the clay content (material finer than 0·002 mm.) increasing with depth. Particle-size curves for samples of soil from different depths are shown in Fig. 21·5, together with the results of index tests. As seen from Fig. 21·4 the top 2 ft of subgrade was on an average slightly wetter than the mean plastic limit.

21·26 The variation in shear strength of the soil under the centre of the road and the theoretical curves for the shear stress produced by a 12,000-lb. wheel load are given in Fig. 21·6. These show that on the basis of the maximum shear stress criterion there was insufficient thickness of construction along the centre of the road.

21·27 The results of C.B.R. tests made on undisturbed samples of soil are given in Table 21·3.

21·28 For a 12,000-lb. wheel load the C.B.R. design charts indicate a thickness of construction of about 25 in., which supports the conclusion that there was an insufficient thickness of construction along the centre of the road.

TABLE 21·3

RESULTS OF C.B.R TESTS ON UNDISTURBED SAMPLES

Description	Average moisture content (%)	Average C.B.R. (%)
Unsoaked specimens	28	4
Soaked specimens	34	2

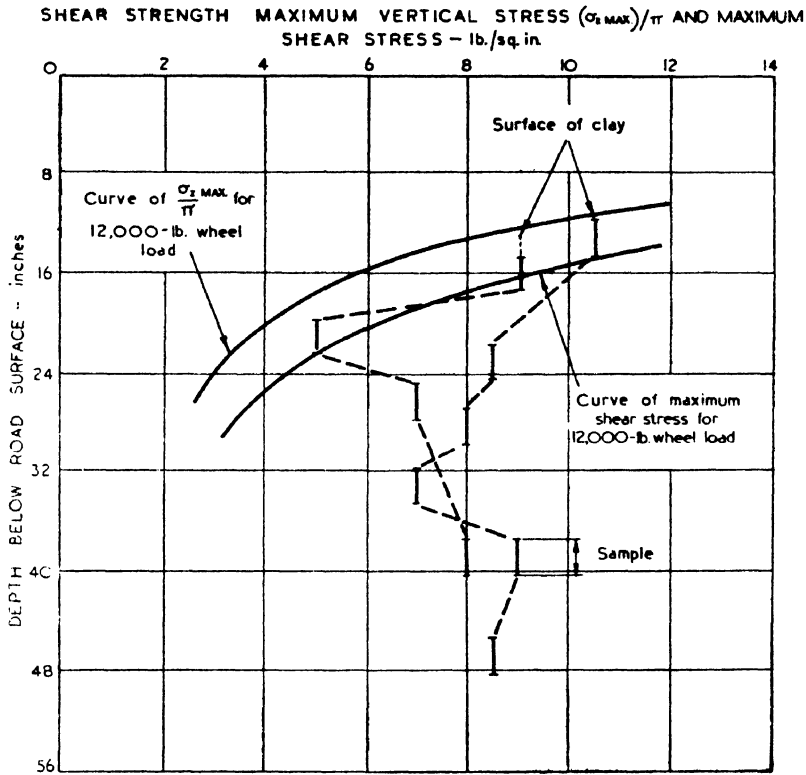



FIG. 21·6 VARIATION WITH DEPTH OF SHEAR STRENGTH OF SUBGRADE OF A FAILED BITUMINOUS ROAD

Clay subgrade. 12,000-lb. wheel load

21·29 During an examination of the subgrade it was noticed that water was seeping into a borehole near the northern edge of the road (see Fig. 21·4), indicating that the 9-in. drain on the northern side of the road was not intercepting all the surface and gravitational subsoil water running off the high ground on that side. This seepage of water into the subgrade was probably a contributory cause of the trouble experienced along the northern edge of the road. For more efficient operation the 9-in. drain should have been located deeper and the drain trench backfilled to ground level with permeable material so as to trap surface runoff.

21·30 The main factors contributing to the failure were, therefore, an insufficient thickness of construction along the crown of the road and an inefficient intercepting drainage system.

An Investigation of Differential Movements between Slabs of a Concrete Road

21·31 The subject of the investigation consisted of a concrete carriageway 20 ft wide with a central tongued longitudinal joint laid on an existing tar-macadam road. Differential movements had occurred along the longitudinal

joint at intervals over a length of about 900 ft of the road, the slabs being as much as 2 in. lower on one side than on the other (Plate 21·4).

21·32 The subgrade consisted of a heavy clay, highly susceptible to volume changes in which the percentage of material finer than 0·002 mm. increased with depth. Particle-size curves and results of the index tests for samples of the soil taken from different depths are given in Fig. 21·7.

Depth (in)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
24	66	27	39
36	89	30	59
48	103	31	72

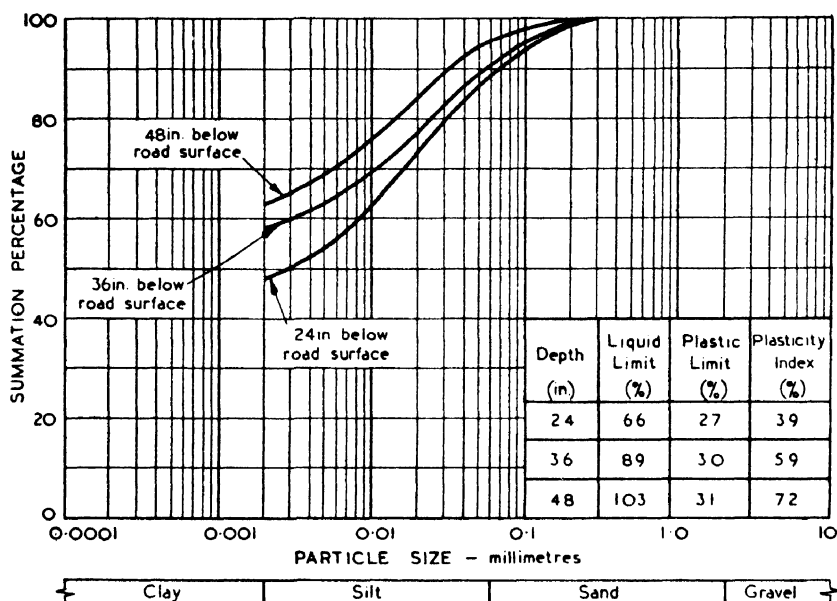



FIG. 21·7 PARTICLE-SIZE CURVES AND RESULTS OF INDEX TESTS OF SOIL FROM SUBGRADE OF CONCRETE ROAD WHICH HAD DEVELOPED SERIOUS DIFFERENTIAL MOVEMENTS BETWEEN SLABS

21·33 The distribution of moisture in the subgrade at one of the sites selected for investigation is shown in Fig. 21·8. The average moisture content of the top 2 ft of subgrade was 5 per cent less under the lower half of the road (south side) than under the other side. This difference in moisture content of the subgrade would be sufficient to account for the differential movement of the two halves of the road, assuming reasonably uniform moisture conditions when the slabs were laid.

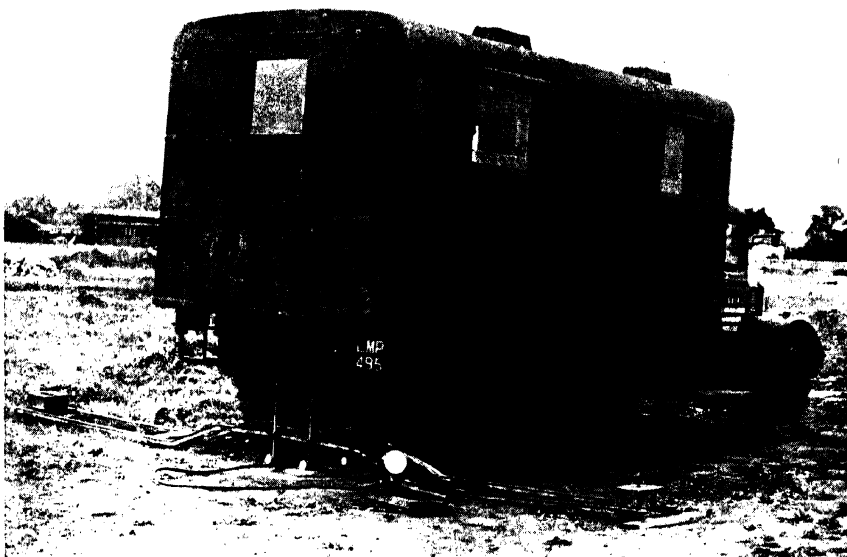
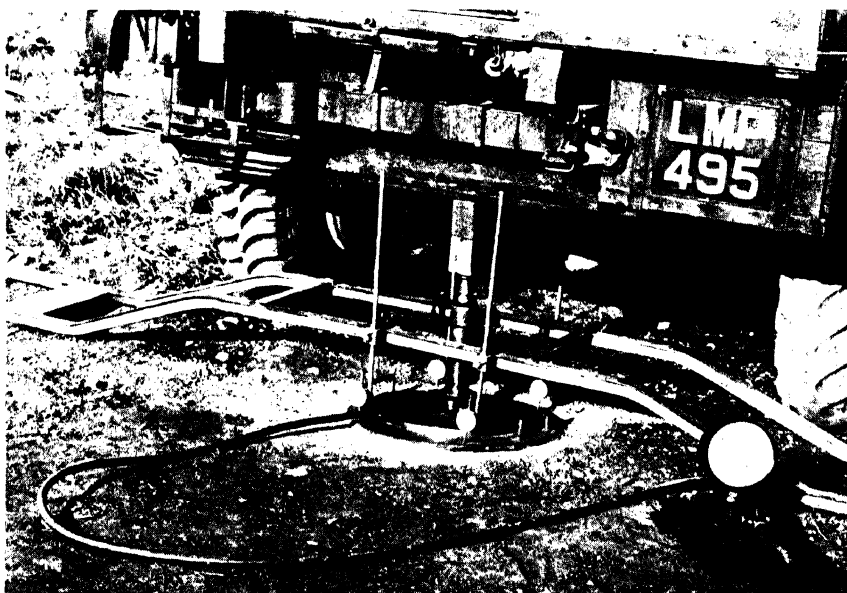
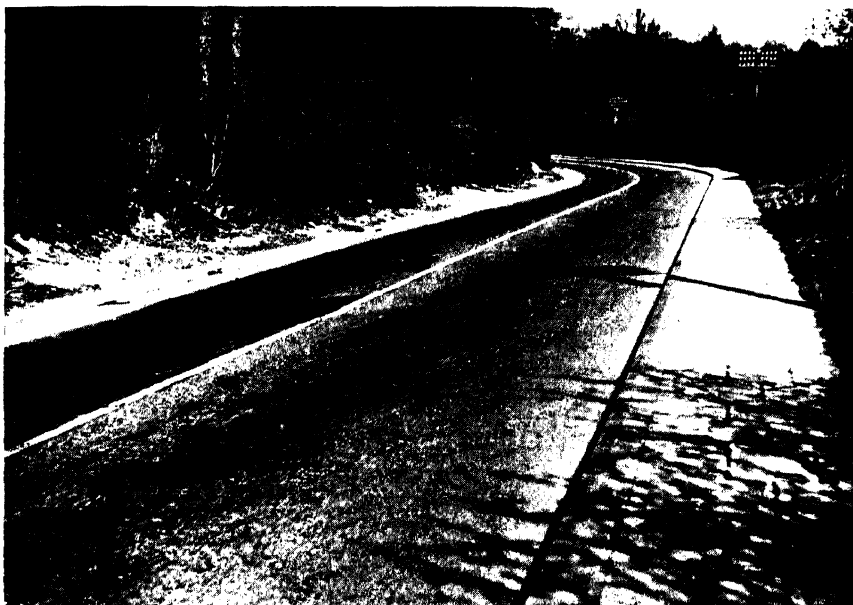
21·34 Further evidence to support the conclusion that volume changes in the clay subgrade were the cause of the differential movements was the fact that, at a site where no movements of the slabs had taken place, no difference was found in the average moisture content of the two halves of the subgrade. In addition, when the road was revisited after two months of wet weather, the edges of the road were found to have risen as much as 2 in., as shown in Fig. 21·8.

(A) EQUIPMENT USED IN INVESTIGATIONS TO MEASURE THE BEARING VALUE OF THE SOIL

(B) CLOSE-UP OF THE PLATE-BEARING TEST APPARATUS

PLATE 21·1

AREA OF CRACKING AND SETTLEMENT OF A CONCRETE ROAD

PLATE 21·2

(A) GENERAL VIEW OF BITUMINOUS-SURFACED ROAD WHICH HAD DEVELOPED AREAS OF SETTLEMENT AND DISINTEGRATION

(B) CLOSE-UP VIEW OF AREA OF SETTLEMENT AND DISINTEGRATION
along the crown of the road

PLATE 21·3

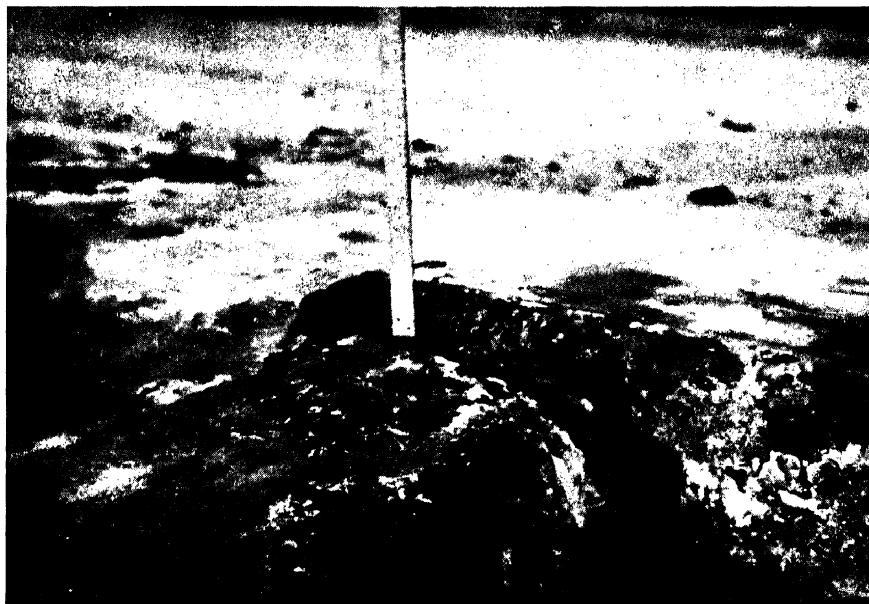
(A) DIFFERENTIAL MOVEMENTS BETWEEN SLABS OF A CONCRETE ROAD

(B) CLOSE-UP VIEW OF DIFFERENTIAL MOVEMENT
(Note displacement of approximately 2-in.)

PLATE 21·4

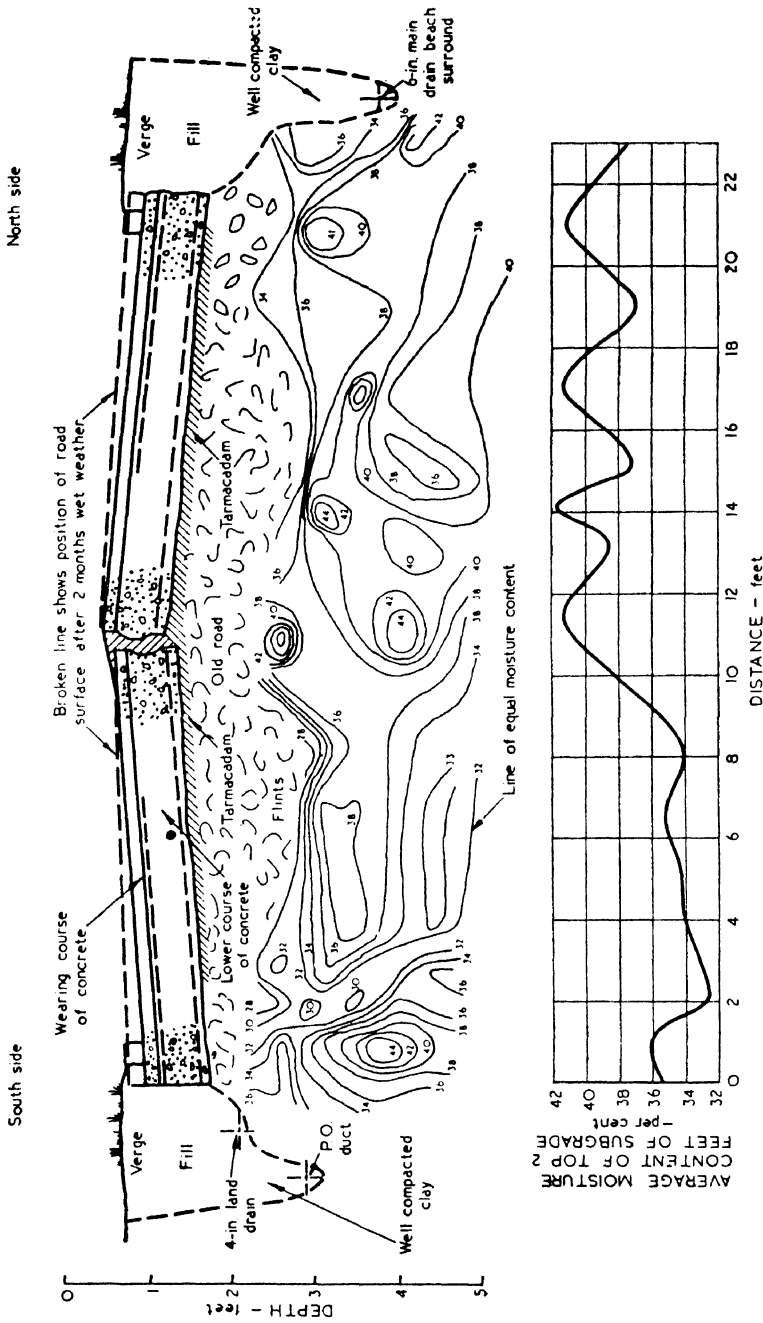



FIG. 21·8 MOISTURE DISTRIBUTION IN SUBGRADE UNDER DISPLACED SLABS

Note higher average moisture content under north half of road

21·35 The possibility that the differential movements might be due to traffic loads is discounted by the extremely large thickness of construction of the road. As shown in Fig. 21·8, this amounted to about 21 in.; the thickness required by the C.B.R. and the shear strength method of design for a 12,000-lb. wheel load was 18·5 in. and 17 in. respectively.

SUMMARY

21·36 In recent years numerous investigations of road foundation failures have been undertaken by the Road Research Laboratory. The primary object is to reveal the causes of the defects and thus to obtain information leading to improved methods of design, but they are, of course, of immediate assistance to the road engineer responsible for the maintenance of the roads.

21·37 The proper understanding of the causes of failure is rendered more difficult because of the absence of information on the conditions obtaining when the failure first developed. To overcome this, as far as practicable, a method of comparing conditions at neighbouring sound and defective areas has been employed.

21·38 At selected sites, trenches about 3 ft wide are cut through the road construction to expose the subgrade. The soil type and the distribution of moisture in the subgrade are determined and the level of the water-table located if it is within 8 ft of the surface. Density measurements are made in the top 9 in. of the subgrade and values of relative compaction are calculated using the results of B.S. compaction tests. In the case of clay soils, the unconfined compression test is used to determine the variation of the shear strength of the soil with depth. California bearing ratio tests are made on undisturbed samples of the subgrade and at each site plate-bearing tests are made on the exposed subgrade and the modulus of subgrade reaction (" k ") is measured. The field work is carried out by a small team of assistants using a mobile laboratory stationed at the site during the investigation.

21·39 The application of the technique is illustrated by three investigations of road foundation failures.

CHAPTER 22

STRESSES IN SOILS AND BEARING CAPACITY OF GROUND

INTRODUCTION

22·1 A knowledge of the distribution of stresses and of the stress/strain relationships in the component parts of a road structure is of fundamental importance, in particular to pavement design and to estimating settlement due to the consolidation of soil. The purpose of this chapter is to provide an introduction to the theories of elastic and plastic behaviour of soils, the application of which is more fully described in Chapters 20 and 23. In the theories of both the elastic and plastic behaviour of soil assumptions are made, the validity of which may be open to question. However, in the absence of theories that are more truly representative of soils as engineering materials, they provide a useful working basis and have been found to give quite good agreement with practice.

CRITERIA OF ULTIMATE FAILURE OF SOIL

22·2 There are a number of possible criteria of ultimate failure of soil. At various times, it has been suggested that failure is governed by maximum principal stress, by maximum shear stress, by maximum total strain energy and by maximum shear strain energy. The most widely accepted criterion now is that of the combination of normal and shear stress due to Coulomb:—

$$s = c + \sigma_n \tan \phi$$

where s = shear strength of material

c = apparent cohesion of material

σ_n = normal stress on sheared face

ϕ = angle of shearing resistance of material.

22·3 The meaning of this criterion can best be appreciated with the aid of the Mohr circles of stress. If test specimens of a soil are subjected to different states of complex and simple stress, for each mode of failure the Mohr circle can be constructed. If a straight line envelope be drawn to the set of Mohr circles so obtained it will be of the form of Coulomb's equation with the apparent cohesion of the soil being given by the intercept of the envelope with the (vertical) shear axis, and the angle of shearing resistance by its slope (see Fig. 22·1). The criterion is that if a Mohr circle, representing the state of stress at a point in the soil, cuts the envelope then failure will take place at that point. It will be seen from Fig. 22·1 that large compressive stresses can be withstood by the soil, provided that they are related as required by this criterion. For some materials, the envelope drawn to fit the Mohr circles derived from test results is not a straight line, but for practical purposes it can be assumed to be straight for soils. This is one of the fundamental assumptions in Prandtl's analysis of plastic failure described below.

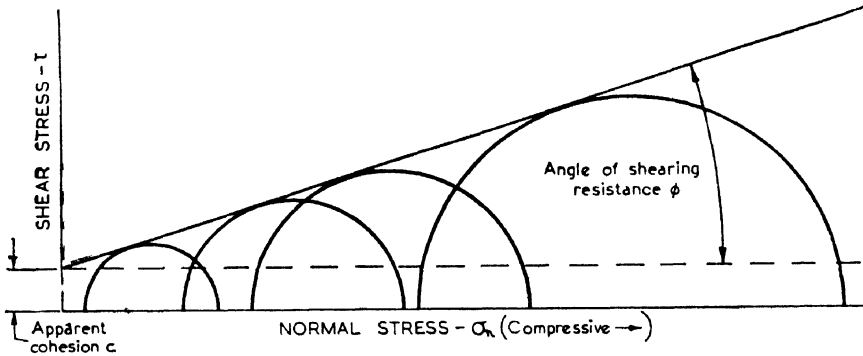



FIG. 22·1 ILLUSTRATION OF COULOMB'S LAW BY MEANS OF THE MOHR CIRCLES OF STRESS

THEORIES OF PLASTIC FAILURE OF SOIL

22·4 Prandtl, Terzaghi and Housel have developed theories for estimating the maximum load that ground is capable of supporting. These theories are based on an analysis of the stress conditions for the ultimate plastic failure of the soil. The load causing ultimate plastic failure of soil is usually greater than that causing an elastic failure in which a certain amount of plastic yield takes place. At the ultimate failure of soil, the deformation increases considerably, and in the case of a foundation, for example, produces a sudden settlement accompanied by a heaving of the soil around the foundation.

22·5 The accuracy and value of the formulæ derived from these theories for bearing capacity depend on the extent to which the assumed shape of the surfaces of failure approach reality. It is in this respect that Prandtl's analysis is considered to be the most reliable, since his assumed mode of failure agrees quite well with observations made on both granular and cohesive soils.

Prandtl's Analysis

22·6 Prandtl[1] derived the bearing capacity of a soil having cohesion and internal friction under a uniform strip load. Fig. 22·2, which is a cross-section of the loaded medium, shows the mode of failure considered by Prandtl. The following assumptions are made:—

(1) The soil is homogeneous, isotropic and weightless.

(2) Wedges AFG, ABC, BED, in Fig. 22·2, behave as rigid wedges and move bodily with no deformation. Sectors AFC and BCD deform plastically. The remainder of the medium is essentially unaffected by the load.

NOTE: The actual angles and lengths in the diagram are not assumed but are derived in the course of the analysis.

(3) The line of rupture (envelope to the Mohr circles) for the soil is a straight line.

(4) In the plastic sectors AFC and BCD the stresses along any radius vector such as BX are constant but they vary from radius vector to radius vector, i.e. with the angle ξ.

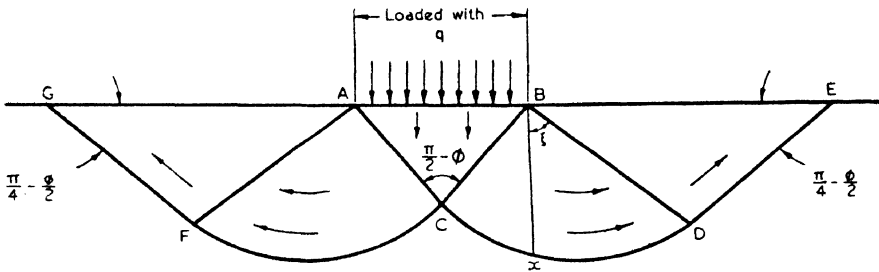



FIG. 22·2 MODE OF ULTIMATE FAILURE ASSUMED BY PRANDTL

22·7 Prandtl considers the equilibrium of the plastic sectors. The boundary conditions are: that the major principal stress on the boundaries AC and BC is q_u, while the minor principal stress on the boundaries AF and BD is zero. If the soil has an apparent cohesion, c, and an angle of shearing resistance ϕ, Prandtl shows that its ultimate bearing capacity is given by

$$q_u = \frac{c}{\tan\phi}\left[\frac{1+\sin\phi}{1-\sin\phi} \,.\, e^{\pi\tan\phi} - 1\right]$$

22·8 It will be seen that, if c = 0, q_u also = 0. This would mean that a cohesionless soil such as dry sand has no bearing capacity. Actually this is not so, and the assumption chiefly responsible for the discrepancy is that the soil is weightless. There are two alternative corrections to Prandtl's formula due to Terzaghi and Taylor respectively. Of these the first is preferred for accuracy but the second is much more easy to calculate.

Terzaghi's Correction for the Weight of the Material

22·9 To c in the original formula add c′
where:—

$$c' = h\,\gamma_b \tan\phi$$

$$h = \frac{\text{area of wedges and sectors}}{\text{length GE}}$$

and γ_b = bulk density of material

The formula then becomes:—

$$q_u = \frac{c+c'}{\tan\phi}\left[\frac{1+\sin\phi}{1-\sin\phi} \,.\, e^{\pi\tan\phi} - 1\right]$$

Taylor's Correction for the Weight of the Material

22·10 To $\frac{c}{\tan\phi}$ in the original formula add:—

$$b\,\gamma_b \cot\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

where:—2b = width of loaded strip
γ_b = bulk density of the material.

The formula then becomes:—

$$q_u = \left[c\,.\,\text{co}\;\phi + b.\,\gamma_b \cot\left(\frac{\pi}{4} - \frac{\phi}{2}\right)\right]\left[\frac{1 + \sin\phi}{1 - \sin\phi}\,.\;^{\pi \tan\phi} - 1\right]$$

Effect of Surcharge on Prandtl's Formula

22·11 It is well known that granular soils are more capable of carrying load if they are confined than if they are not. In the California bearing ratio test, for example, surcharge weights representing the weight of pavement placed round the test plunger produce significant increases in the strength of granular soils.

22·12 If a vertical pressure, p, is applied to the outer rigid wedges AFG and BDE, then the bearing capacity is increased from that given by the preceding Prandtl formula by an additional:—

$$p\left[\frac{1 + \sin.\phi}{1 - \sin\phi}\,.\,e^{\pi \tan\phi}\right] = \lambda p$$

The addition is seen to depend only on ϕ, and Fig. 22·3 shows how the factor in the square brackets, λ, varies with ϕ. This surcharge effect can only apply

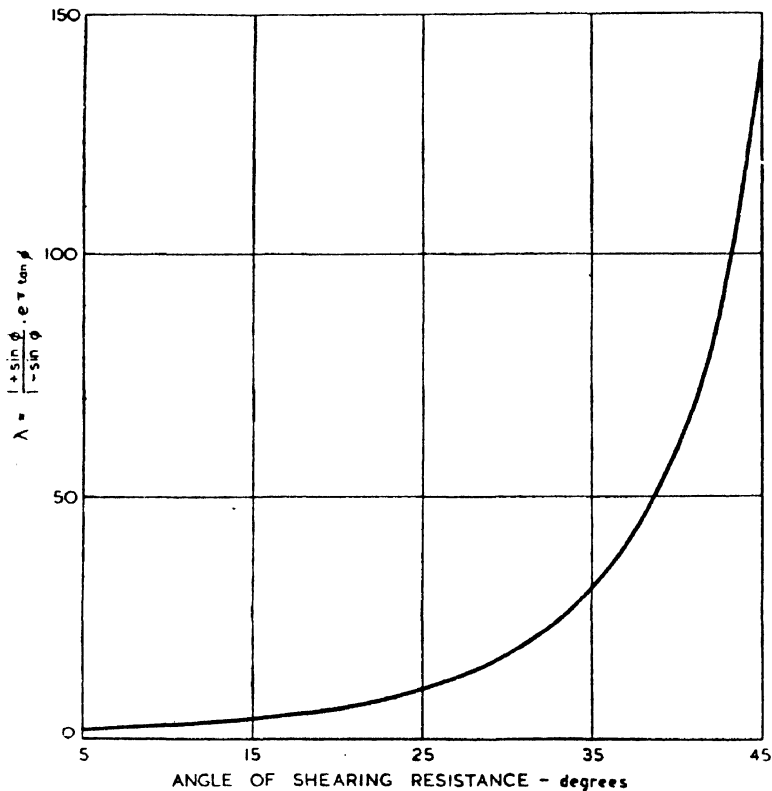



FIG. 22·3 EFFECT OF ANGLE OF SHEARING RESISTANCE ON COEFFICIENT OF INCREASE IN BEARING CAPACITY DUE TO SURCHARGE

for values of p considerably less than q_u for otherwise there results merely an increase in the loaded width. In practice p will usually be small compared with q_u but nevertheless on granular soils, where ϕ may lie between 35° and 40°, marked increase in bearing capacity is produced. On cohesive soils, where ϕ is of the order of 0° to 10°, surcharge produces little effect.

Terzaghi's Analysis

22·13 The mode of failure under a uniform strip load assumed by Terzaghi is shown in Fig. 22·4, which is a cross-section of the loaded medium. He assumes that the soil is homogeneous and considers the stability of the wedges ABD, BDF, ACE and CEG. He considers the important properties of the soil to be the bulk density, the apparent cohesion and the angle of shearing resistance and shows that the bearing capacity is given by:—

$$q_u = \frac{b\,\gamma_b\,(1 - \tan^4 \beta)}{2 \tan^5 \beta} + \frac{2c}{\tan \beta \sin^2 \beta}$$

where γ_b, b and c are as previously defined:—

$$\text{and } \beta = \left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

FIG. 22·4 MODE OF ULTIMATE FAILURE ASSUMED BY TERZAGHI

22·14 As for the modified Prandtl formula, the bearing capacity is a function of the width of the loaded strip, since Terzaghi has assumed from the beginning that the soil has weight. The importance of the width of load is marked on granular, but small on cohesive soils.

Effect of Surcharge on Terzaghi's formula

22·15 If a surcharge, p, is applied on either side of the strip load to the wedges FBD and CEG the bearing capacity is increased by an additional:—

$$p.\ \cot^4 \beta$$

The remarks already made on the relationship between p and q apply equally in this case.

Housel's Perimeter Shear Theory

22·16 In Housel's theory it is suggested that, in addition to the bearing capacity that may be calculated from any of the preceding formulæ, allowance should be made for the shear which takes place round the perimeter of the loaded area when failure occurs, i.e. that the total bearing capacity should be written:—

$$q'_u = q_u + \frac{L}{A} . S$$

where q'_u = the total bearing capacity
q_u = the bearing capacity estimated from one of the preceding formulæ or a modification of it
L = the perimeter of loaded area
A = the loaded area
S = shear on perimeter (force/unit length).

22·17 It has been found experimentally[2] that on soft clays the ratio of the perimeter to the area has an important effect on the apparent bearing capacity. Skempton[3] has also suggested a formula of Housel's type.

22·18 These treatments of the plastic failure of soil are important in estimating the stability of structures and embankments. Usually a factor of safety is chosen by the engineer to reduce the working load below those which these theories would indicate as the ultimate bearing capacity.

CRITERION OF ELASTIC BEHAVIOUR OF SOIL

22·19 The most important characteristic of the elastic behaviour of soil is that no matter how many repetitions of load are applied to it, provided that the stresses set up in the soil do not exceed the " yield stresses " the soil does not become permanently deformed and recovers its original form immediately the load is removed. Under static conditions it is unlikely that any soil behaves quite in this way but under dynamic conditions and especially with cohesive soils such behaviour may occur.

22·20 The treatment of the problem of an elastic medium subjected to any specified loading depends in its analysis on the distribution of stress beneath a point load.

THEORY OF ELASTIC BEHAVIOUR OF SOIL

Boussinesq's Analysis

22·21 The distribution of the stresses in a semi-infinite, homogeneous, isotropic, elastic medium due to the application of a vertical point load was first derived by Boussinesq[4] The first difficulty that arises is that since the contact area under a point load is zero the stresses must be infinite and elastic and plastic failure must occur. Boussinesq argued that once flow had taken place the surrounding material would experience only finite stresses. He therefore replaced the point load by a hydrostatic pressure acting over the surface of a small hemisphere in the medium with its centre at the loaded point. Fig. 22·5 shows

the case considered and with the nomenclature of that figure the stress distribution is such that:—

$$\text{the vertical stress, } \sigma_z = -\frac{3P}{2\pi} z^3 \left(x^2 + y^2 + z^2\right)^{-\frac{5}{2}}$$

$$\text{the shear stress } \tau_{rz} = -\frac{3P}{2\pi} rz^2 \left(x^2 + y^2 + z^2\right)^{-\frac{5}{2}}$$

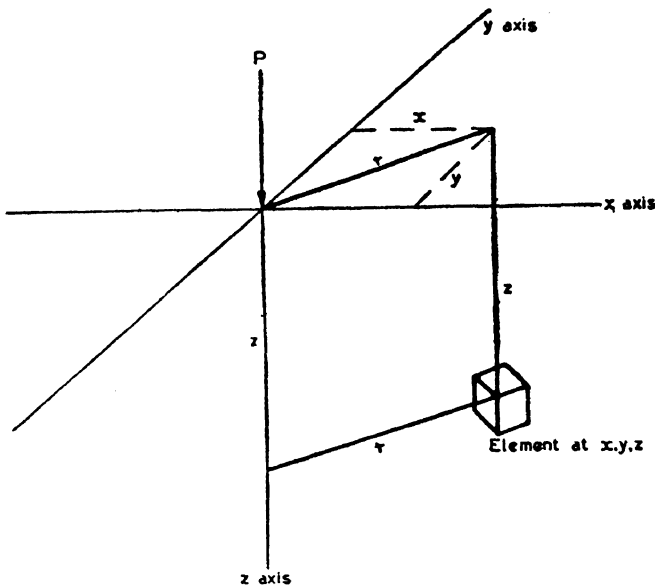
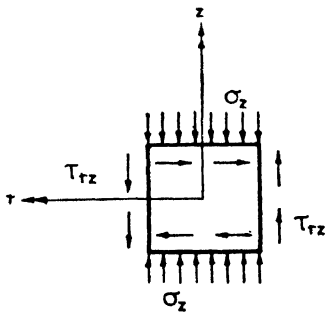
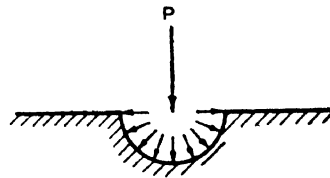
$$\text{where } r^2 = x^2 + y^2$$

(a) Position of element with respect to the point load P

(b) Vertical and shear stresses on element

(c) Assumed hydrostatic pressure on a hemisphere is equivalent to P

FIG. 22·5 STRESSES UNDER POINT LOAD (BOUSSINESQ ANALYSIS)

22·22 Expressions for the radial and circumferential (horizontal) stresses are given by Timoshenko[5]. However, in most problems of settlement and pavement design, it is only required to know the vertical and shear stresses. In Fig. 22·6 is shown the distribution of vertical stress on horizontal planes beneath a point load. On each plane there is a symmetrical bell-shaped distribution and the peak height of the bell decreases as the square of the distance of the plane from the loaded surface increases. Moreover, if any line is drawn from the loaded point at an angle to the axis of symmetry it will cut these horizontal planes at a series of points at which the vertical stress also decreases as the square of the distance of the plane from the loaded surface increases. Those points which experience the same vertical stress can be joined by curves and the resulting family of iso-stress surfaces forms a " pressure bulb." The " pressure bulb " for a point load is shown in Fig. 22·7.

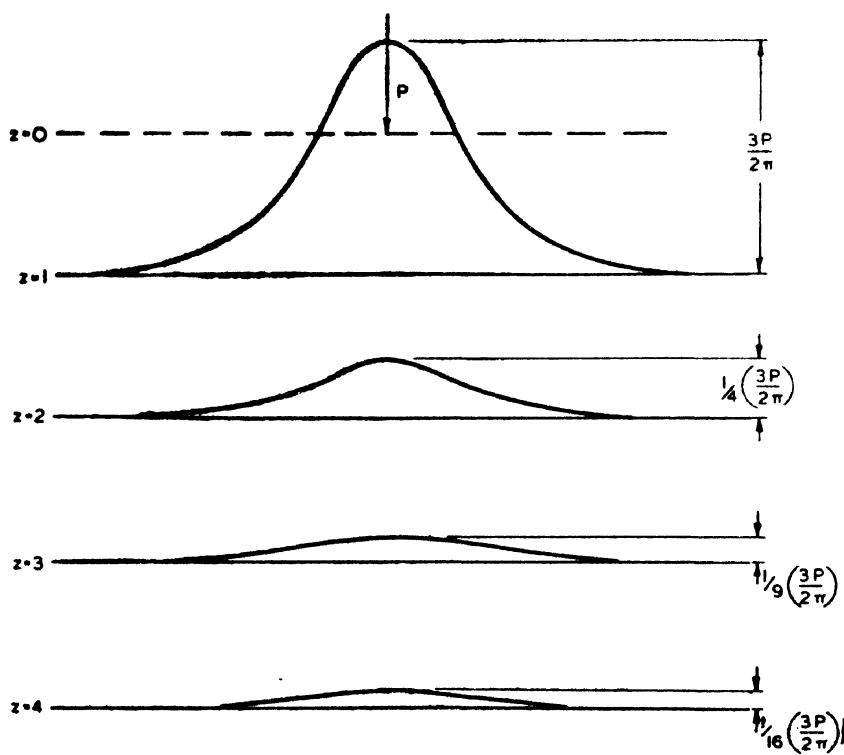



FIG. 22·6 DISTRIBUTION OF VERTICAL STRESS ON HORIZONTAL PLANES BENEATH A POINT LOAD

22·23 Another interesting fact is that, for points on any sphere drawn in the medium tangentially to the loaded point, the total resultant stress on horizontal faces at these points is constant and is equal to

$$\frac{3P}{2\pi d^2}$$

where d is the diameter of the sphere.

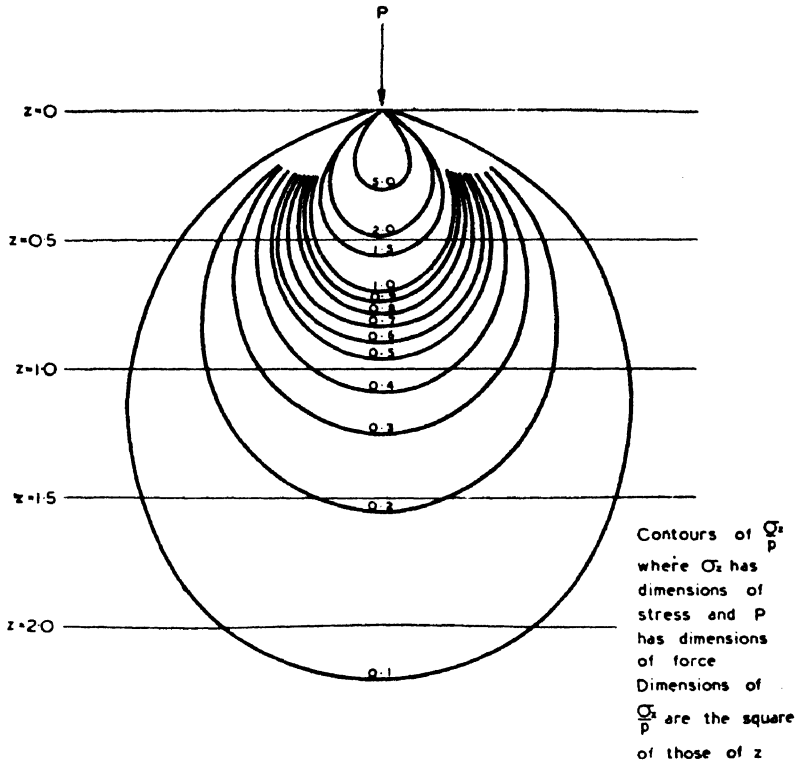



FIG. 22·7 BULBS OF PRESSURE UNDER POINT LOAD

22·24 Having obtained the distribution of stress beneath a point load it is possible, by using the principle of superposition and integrating, theoretically to obtain the distribution of stress beneath any loading system. Consider a point beneath a loaded surface. The Boussinesq analysis gives the stresses at this point due to any elementary point load. Hence, by summation, the stresses at the point due to all the elementary loads which go to make up the load system can be calculated. This usually entails a double integration of an expression such as that quoted for σ_z between limits whose complexity depends on the shape of the loaded area.

22·25 In practice, a complete solution can only be obtained for a relatively few simple shapes of loaded area, such as line, strip, rectangle, square and circle, under load distributions which are either simple or vary in a uniform manner. Some of these have been calculated by Jürgenson(6), and influence chart methods have been suggested by Terzaghi(7) and Newmark(8) for more difficult shapes of loaded area. All of these cases cannot be dealt with here but a few examples may be of interest.

Uniform Strip Load

22·26 This is the same loading system as that considered by Prandtl and forms an essentially two-dimensional problem. Thus it is only necessary to consider a slice of the medium with a line load on the upper edge.

22·27 The "pressure bulb" for this case is shown in Fig 22·8 and the distribution of shear stress is shown in Fig. 22·9. The maximum shear stress, whose intensity is $1/\pi$ times the applied pressure, occurs on a semicircle with the loaded width as diameter. This is the basis of one method of pavement design.

22·28 The distribution of stress beneath a uniform strip load is important in estimating settlements of structures and embankments.

Uniform Circular Load

22·29 The "pressure bulb" for a uniform circular load is shown in Fig. 22·10. By comparing this with Fig. 22·8, it will be seen that the stresses are less intense under a uniform circular load than they are under a uniform strip load of the same width.

22·30 The case of the circular load is used in some methods of pavement design.

Strip with Triangular Loading

22·31 The "pressure bulb" for the case of triangular loading is shown in Fig. 22·11. This is of use in estimating the settlement of embankments. A normal section of an embankment may be considered as the difference between two triangles of equal angles but unequal length of side. Hence the stress distribution beneath such an embankment may be obtained by subtracting the stresses due to the smaller triangle from those due to the larger triangle.

Uniform Rectangular Load

22·32 The vertical stress beneath a corner of a uniformly loaded rectangle can be expressed as

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\,(m^2+n^2+1)^{\frac{1}{2}}}{m^2+n^2+m^2n^2+1}\cdot\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\,(m^2+n^2+1)^{\frac{1}{2}}}{m^2+n^2+1-m^2n^2}\right]$$

where q = the intensity of the uniform load

$m = \dfrac{B}{z}$

$n = \dfrac{L}{z}$

B = breadth of the rectangle

L = length of the rectangle

z = depth of point beneath corner of rectangle.

(Note: L and B, or m and n are interchangeable.)

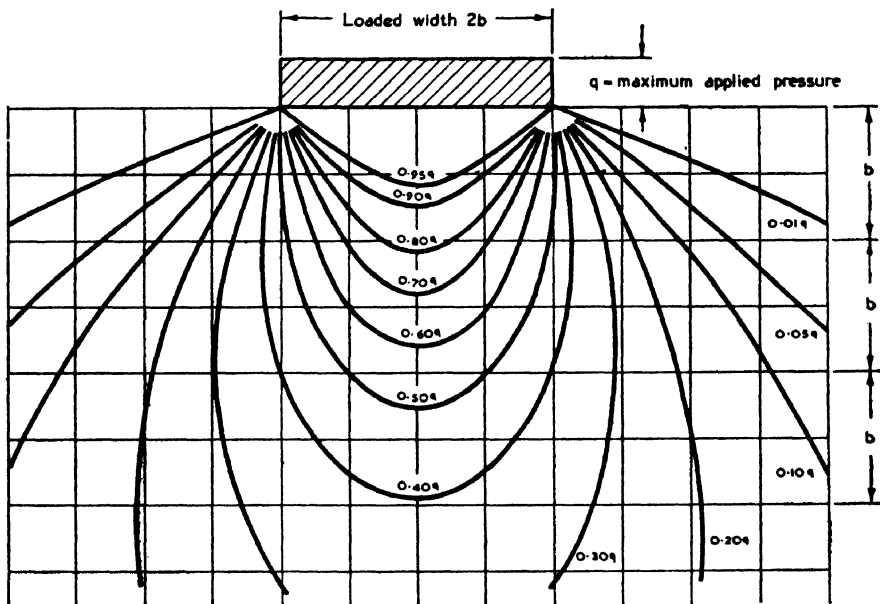



FIG. 22·8 BULB OF PRESSURE UNDER UNIFORM STRIP LOAD

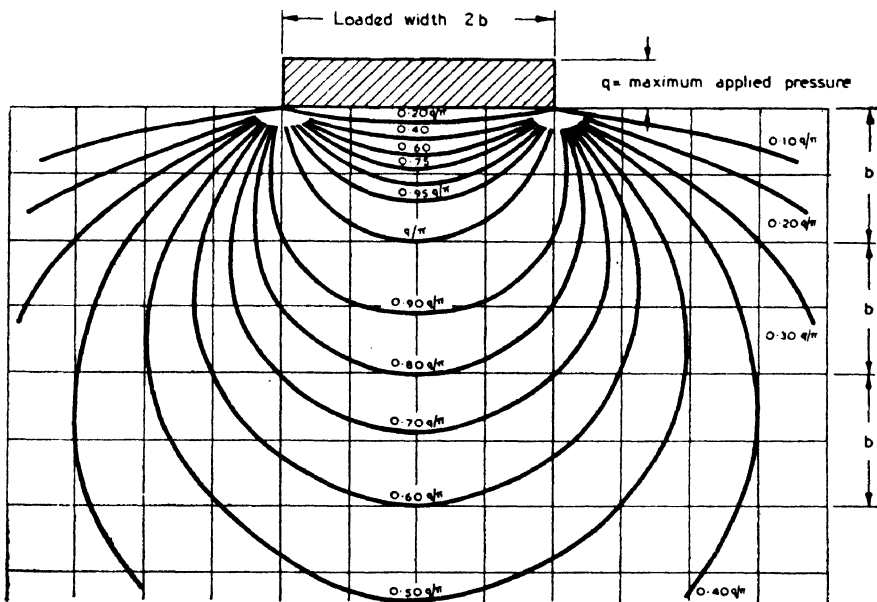



FIG. 22·9 DISTRIBUTION OF MAXIMUM SHEAR STRESS UNDER UNIFORM STRIP LOAD

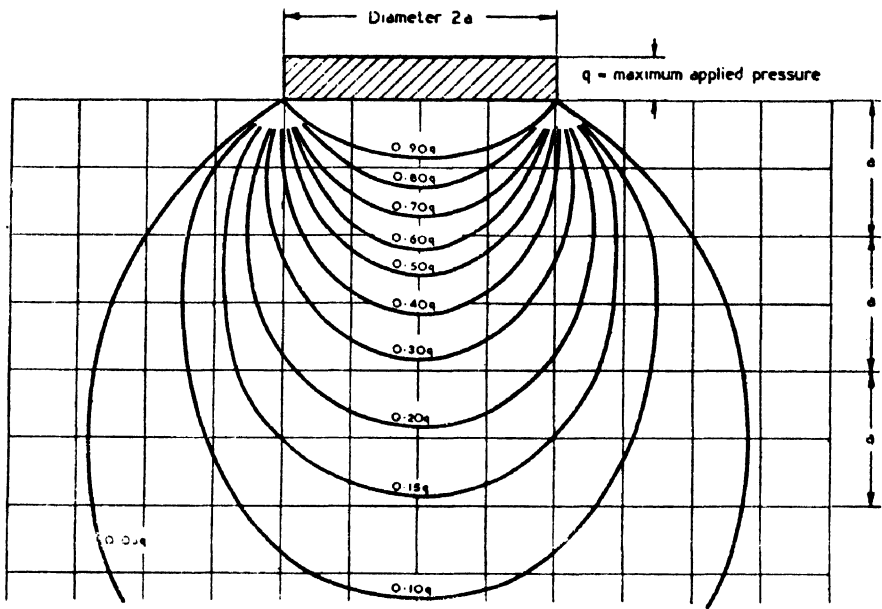



FIG. 22·10 BULB OF PRESSURE UNDER UNIFORM CIRCULAR LOAD

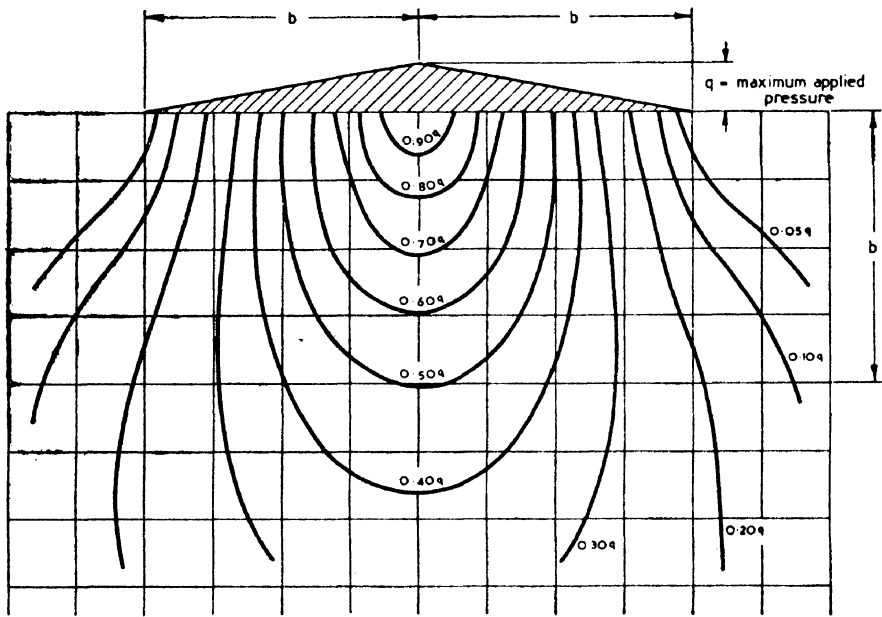



FIG. 22·11 BULB OF PRESSURE UNDER STRIP WITH TRIANGULAR LOAD

22·33 A rectangle can be divided into, say, four smaller rectangles by drawing lines parallel to the sides of the original rectangle through any given point. Hence, the stress at any point beneath a loaded rectangle can be considered as the sum of four stresses due respectively to the four rectangles whose corners coincide above the point in question.

22·34 Hence, if any loaded area can be divided into rectangles, the distribution of vertical stress beneath it may be obtained by adding and subtracting the values of σ_z for the constituent rectangles (re-entrants counting as negative).

22·35 This is of importance in estimating the settlement under irregularly shaped foundations of structures and is the basis of the influence chart methods referred to above.

SUMMARY

22·36 This chapter gives an introduction to the methods of estimating safe loads on soils and the distribution of stresses in soils. These methods are different according to whether the soil is considered to be elastic or plastic, and are of importance, for example, in designing pavement thickness and in estimating settlements due to consolidation.

22·37 A brief account is given of the treatment of an elastic medium following the method of Boussinesq and of the treatment of a plastic medium by Prandtl and others. Some examples of the distribution of stress in an elastic medium under various loading conditions are given together with references to sources of further information.

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CHAPTER 23

THE CONSOLIDATION OF COMPRESSIBLE SOILS

INTRODUCTION

23·1 The consolidation of compressible soils is largely the concern of the foundation engineer but the road engineer encounters consolidation problems in the construction of embankments, bridge abutments and fly-over junctions. The settlement of such structures would not in itself be very serious if the movements were always uniform, but unfortunately they are rarely so and the differential settlements which may occur can result in the development of additional stresses which may greatly exceed the design stresses. Thus, differential settlements in the case of concrete roads founded on embankments may result in serious cracking of the slabs while differential settlements of bridge abutments or the foundation of fly-over junctions may cause failure of the entire structure. Peat soils, being highly compressible and usually varying in nature and thickness, can cause serious differential settlements of roads even in cases where the loading is uniform and relatively small. During an inspection of roads constructed on peat, certain lengths were seen to be badly deformed (Plate 23·1). In this particular instance it was considered that the differential nature of the settlement was due to the presence of tree trunks in the peat. Serious differential settlements can also occur where culverts and other rigid structures underlie the road (Plate 23·2).

23·2 Terzaghi and others have developed a general theory whereby the settlement of any structure founded on compressible soil can be computed and it is now possible to predict the settlement of a proposed structure to quite a fair order of accuracy and so modify the design, if necessary, to reduce the effects of serious differential movements.

23·3 Perhaps the earliest example of the differential settlement of a structure is the famous leaning tower of Pisa. Construction of the tower began in 1174 A.D. and soon afterwards differential settlement of the foundation occurred which continued until at present there is a tilt of 16 ft in its height of 179 ft. Modern investigations have indicated that the movements are probably due to consolidation in a thick layer of clay underlying the tower.

23·4 This chapter gives the essentials of Terzaghi's mathematical theory of consolidation and shows how it is applied to structures in general.

Definitions

23·5 The terms used in this chapter are defined in Chapter 28. The term consolidation refers to the reduction in volume of a saturated soil when acted on by a static load over a long period of time. Undisturbed clay soils only contain a small amount of air voids and can normally be considered as saturated. The consolidation of a soil is usually measured in terms of the reduction of its voids ratio.

23·6 In the case of a saturated soil, the voids ratio is directly proportional to the moisture content for:—

$$\text{Voids ratio} = \frac{\text{Volume of voids (water)}}{\text{Volume of soil particles}}$$

$$= \frac{\text{Weight of water}}{\text{Weight of soil particles}} \times \frac{\gamma_s}{\gamma_w}$$

$$= \text{moisture content} \times \frac{\gamma_s}{\gamma_w}$$

where γ_s = weight per unit volume of soil particles

γ_w = weight per unit volume of water.

PROCESS OF CONSOLIDATION

23·7 The process of consolidation is best illustrated by reference to the model analogy shown in Fig. 23·1, which has been employed by Terzaghi when demonstrating the fundamentals of consolidation. This model consists of a cylinder with a closely fitting piston perforated with a number of small holes resting on a number of compression springs. The cylinder beneath the piston is filled with water. Consider what occurs when a vertical load is applied suddenly to the piston. Instantaneously, all the load is carried by the water, the springs being far more compressible than the water. The pressure in the water immediately under the piston thus equals the applied pressure. Owing to the difference between the pressure existing in the water and that in the atmosphere, water will commence to flow out through the holes in the piston, the rate of flow depending on the difference of pressure. As water escapes through the piston a reduction in the water pressure will ensue and the load

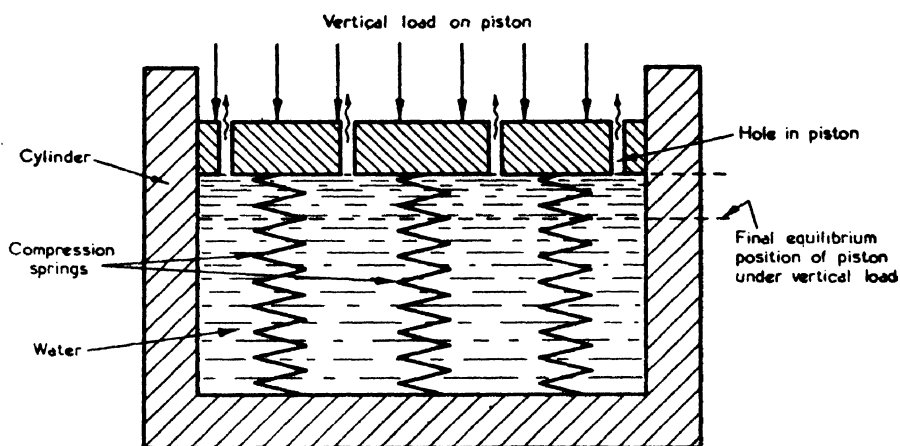



FIG. 23·1 TERZAGHI'S MODEL ANALOGY, ILLUSTRATING THE PROCESS OF CONSOLIDATION

will be transferred gradually to the springs. Eventually, the pressure difference of the water will reduce to zero and all the load will be carried by the springs, the piston coming to rest in a new equilibrium position. It will be seen that the speed with which the process takes place will depend upon the number and size of the holes in the piston.

23·8 Turning from this analogy to the case of soils: when a load is applied to a saturated soil, the entire load is at first carried by the water in the voids. The pore-water pressure thus equals the applied pressure. If the soil is very permeable, the increase in the pore-water pressure under the loaded area will cause a rapid flow of water out of the voids until the entire load is carried by the structure formed by the soil particles. If the soil is impermeable then the water will flow out very slowly; this is the reason why the process of consolidation in clays, for example, extends over a considerable period of time. The complete process may require many hundreds of years. The rate of the consolidation process is relatively rapid at first but decreases with time.

23·9 Although a knowledge of the total settlement of a proposed structure founded on a compressible soil is valuable, of far greater interest and importance is a knowledge of the rates of settlement during the early years of the life of a structure.

MATHEMATICAL THEORY OF CONSOLIDATION

23·10 A mathematical theory of consolidation has been worked out by Terzaghi many years ago and is given in some detail in his book " Theoretical Soil Mechanics "[1]. The theory is fairly complex and the final differential equation that Terzaghi obtained for the process of consolidation can only be solved by means of an infinite series.

23·11 The main assumptions which Terzaghi made are as follows:—

(1) The voids of the soil are completely filled with water. That is, the soil is saturated.

(2) The water and the soil particles are incompressible.

(3) Darcy's Law for the velocity of flow of water through soil is perfectly valid, i.e. velocity = K.i, where K is the coefficient of permeability and i is the hydraulic gradient.

(4) The coefficient of permeability, K, is a constant during the process.

(5) The time lag of consolidation is due entirely to the low permeability of the soil.

(6) The soil is laterally confined.

(7) The total and effective normal stresses are the same for every point on any horizontal plane. That is, the flow of water out of the voids takes place only in a vertical direction.

(8) The change in effective pressure in the soil causes a corresponding change in voids ratio.

Validity of the Assumptions in relation to Practical Conditions

23·12 The assumption that the soil is in a saturated condition is sufficiently true in the case of undisturbed compressible clay soils situated below the top

4 to 5 ft of surface soil. The fact that the water and soil particles are considered to be incompressible means that the settlement due to the elastic compression of the soil is neglected in comparison with the settlement due to consolidation. This is perfectly legitimate in most cases that are met with in practice. Darcy's Law has been found to agree reasonably closely with experimental data but the assumption that the coefficient of permeability is constant during the process is open to question. The assumption that the soil is laterally confined and especially the assumption that the water flows out of the soil only in a vertical direction probably introduces the greatest error. However, when the area of soil that is loaded is large, the error due to this assumption may be small.

23·13 Although it may appear that the assumptions made are not very close to the truth, in many cases the predicted settlements agree reasonably well with the actual measured settlements.

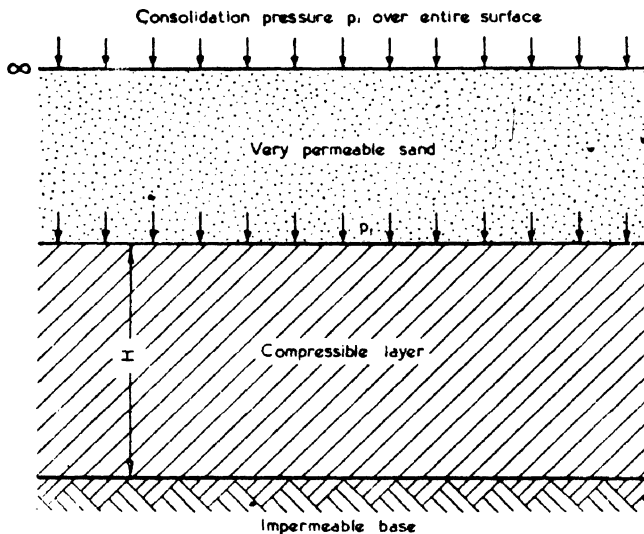



FIG. 23·2 BOUNDARY CONDITIONS USED IN TERZAGHI'S MATHEMATICAL THEORY OF CONSOLIDATION

23·14 On the basis of the assumptions given above and the boundary conditions shown in Fig. 23·2, Terzaghi obtained the following equation relating the settlement ρ at any time t after the sudden application of a uniform pressure to the surface of the soil:—

$$\rho = m_v \, p_1 \, H \left[1 - \frac{8}{\pi^2} \sum_{N=0}^{N=\infty} \frac{1}{(2N+1)^2} e^{-(2N+1)^2 \frac{\pi^2}{4} \frac{C_v t}{H^2}} \right] \quad (1)$$

where m_v = coefficient of volume change

$$= \frac{\Delta e}{\Delta p} \times \frac{1}{(1 + e_1)}$$

where Δe = change in voids ratio during the process

Δp = change in the pressure causing the change in voids ratio

e_1 = initial voids ratio before p_1 is applied

p_1 = uniform pressure applied to the surface of the soil (See Fig. 23·2)

H = thickness of the consolidating layer in the case considered. (See Fig. 23·2.)

H is in fact the length of the drainage path, the minimum distance which the water particles situated furthest from the free drainage layer have to travel to reach that layer.

C_v = coefficient of consolidation $= \dfrac{K}{\gamma_w m_v}$

where K = coefficient of permeability

γ_w = weight per unit volume of water.

e = base of the Napierian system of logarithms.

23·15 In equation (1) $m_v p_1 H$ represents the total settlement which would occur after an infinite period of time. $\dfrac{C_v t}{H^2}$ is a dimensionless term known as the time factor, t_v.

23·16 The expression for settlement (equation (1)) thus consists of a term representing the final settlement and a term in brackets which always represents some function $f\left(\dfrac{C_v t}{H^2}\right)$, of the time factor.

Thus, the expression for settlement can be written:—

$$\rho = \rho_1 f(t_v)$$

where ρ_1 is the ultimate settlement.

The percentage consolidation or degree of consolidation that has occurred at any time, t, is:—

$$U \text{ (per cent)} = \frac{\rho}{\rho_1} \times 100$$

$$= f(t_v) \times 100$$

$$= f\left(\frac{C_v t}{H^2}\right) \times 100$$

This equation shows that for a given soil and degree of consolidation, t is proportional to H^2. That is, the rate of consolidation is inversely proportional to the square of the length of the drainage path. The mathematical theory of consolidation outlined above, although of interest, need not be used in the estimation of the settlement of a structure. The important fact that the rate of consolidation is inversely proportional to the square of the length of the drainage path is used, however, in the calculation of the rate of settlement of

a structure from the results of a small-scale laboratory test and it is for this rea.on that a brief résumé of the theory of consolidation has been included here.

23·17 Those interested in the mathematical details of the theory are advised to consult Terzaghi's book, "Theoretical Soil Mechanics"[1].

LABORATORY CONSOLIDATION TEST

23·18 The rate and degree of settlement of a proposed structure founded on a compressible soil is estimated from the results of small-scale laboratory tests carried out on samples of the underlying soil.

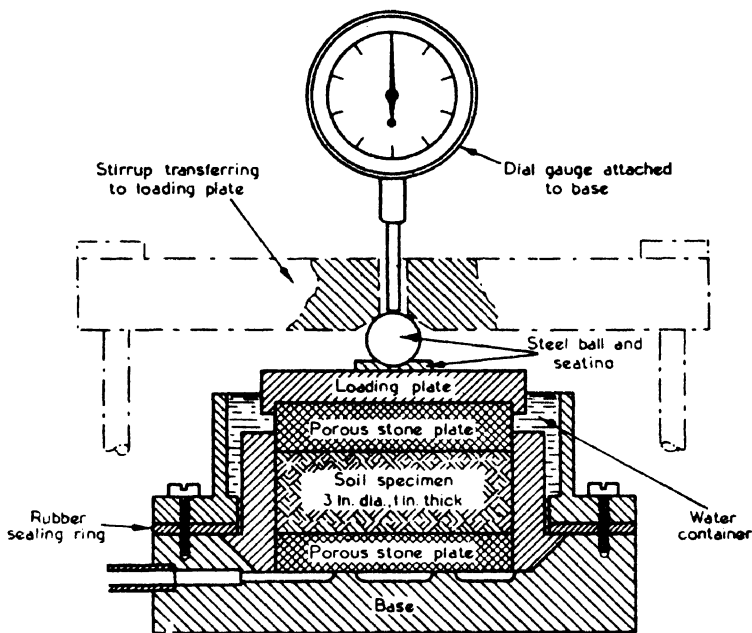



FIG. 23·3 DIAGRAMMATIC CROSS-SECTION OF CONSOLIDATION APPARATUS

23·19 The apparatus used in the laboratory test is shown diagrammatically in Fig. 23·3. The apparatus is so designed and the test is carried out in such a manner that the conditions assumed in the mathematical theory of consolidation are valid. The soil specimen of known thickness is laterally confined in a metal mould and is sandwiched between two porous plates. The size of specimens tested by different laboratories differs considerably. At the Road Research Laboratory a 3-in. diameter specimen with an initial thickness of 1 in. is used for routine measurements. Loading is applied to the specimen through the porous plates using a stirrup and steel ball located centrally over the specimen by means of a loading plate and a ball seating. The change in thickness of the specimen is recorded with a dial gauge, its push rod resting on top of the ball.

23·20 The apparatus used at the Road Research Laboratory for carrying out consolidation tests is shown in Plates 23·3 and 23·4.

23·21 The consolidation test can be divided into two parts:—

(1) Determination of the voids ratio/effective pressure relationship for the soil under examination.

(2) Determination of the percentage or degree of consolidation/time relationship for the soil under examination.

The readings that are required for these two relationships are taken at the same time during the test.

23·22 The experimental details of the laboratory test are as follows:—The sample of soil to be tested is cut carefully so that it fits perfectly within the mould. This is aided by the use of a circular cutter, the inside diameter of which is a few thousandths of an inch greater than the inside diameter of the mould. The actual technique employed to give the most satisfactory results will depend upon the type of apparatus and soil used and therefore no definite rules can be laid down. However, the proper preparation of an undisturbed specimen is a skilled operation, and to reduce the effects of sample disturbance special forms of consolidation apparatus have been devised in which the sample is tested in the sampling cutter by which it has been obtained. Remoulded specimens are usually prepared by pressing the soil into the mould with a spatula instead of cutting it to size, care being taken to avoid trapping air during the process.

23·23 After the specimen has been placed in the mould and the apparatus assembled, a number of successive increments of load are applied to the sample, each increment of load being allowed to act for a time sufficient to allow the settlement to be complete, generally 24 hours. The successive loads usually employed are $\frac{1}{4}$, $\frac{1}{2}$, 1, 2 and 4 tons/sq.ft, but the limit of the final increment will depend upon the actual loadings on the soil beneath the structure. After the application of each increment of load, readings of the dial gauge are taken after $\frac{1}{2}$, 1, 2, 4, 8, 16, ... etc. minutes until the end of the test (usually 24 hours). The practice of doubling the load after each increment and doubling the time before making the next observation results in a better spacing of points on the graphs.

23·24 If a knowledge of the swelling of the soil under decreasing load is not required, the load is removed after the reading for the final increment has been obtained and the soil specimen removed from the mould. The dry weight of the sample is found in the usual way and a determination is made of the specific gravity of the soil particles. From the information thus obtained and the dimensions of the mould it is possible to calculate the initial voids ratio of the specimen and the voids ratio corresponding to each increment of load. The relationship between voids ratio and effective pressure can then be plotted. The percentage consolidation/time relationship is obtained by assuming that at the end of the test for each increment of load, e.g. after 24 hours, the consolidation is complete and equal to 100 per cent. The percentage consolidation for the intermediate intervals of time, $\frac{1}{2}$, 1, 2, 4, 8, 16, ... etc. minutes can then be calculated from the readings of the dial gauge at these times and a curve plotted.

23·25 If one is interested in knowing how much the soil will swell when load is removed, a swelling test is carried out after the readings for the final increment of the loading test have been obtained. In the swelling test, the loads are removed from the specimen in increments, 24 hours being allowed to elapse before each increment is removed, and the dial gauge readings taken as before. When the load has been reduced finally to zero, the soil specimen is removed, dried, weighed and the specific gravity of the soil particles found. A complete voids ratio/effective pressure relationship for both loading and unloading conditions can then be plotted.

23·26 The results of the swelling test are of use in the determination of the upward movement of soil when the overburden load is removed as in the excavation for a cutting or for a deep foundation.

23·27 Fig. 23·4 shows a typical voids ratio/effective pressure relationship for an undisturbed sample of London clay under both loading and unloading conditions. The hysteresis effect should be noted. The corresponding percentage consolidation/time curve for the soil specimen is shown in Fig. 23·5. It will be seen that, for this specimen and sample thickness (0·4 in.), 90 per cent of the consolidation was complete within the initial 60 minutes of the test.

ANALYSIS OF FOUNDATION SETTLEMENTS

23·28 The calculation of the amount and rate of settlement of a foundation requires a knowledge of:—

(1) The thickness, position and nature of the various soil strata underlying the foundation and the free water conditions.
(2) The voids ratio/effective pressure and percentage consolidation/time relationships for undisturbed samples of the compressible soil.
(3) The stress distribution through the soil after the structure has been erected.

23·29 The first of these requirements will be satisfied from the results of boring tests, using lining tubes if necessary. The consolidation characteristics of the underlying compressible soil will be determined from laboratory tests as outlined previously. The determination of the distribution of stress is much more complex, however, and certain simplifying assumptions have to be made.

Distribution of Stresses in Soil

23·30 As shown in Chapter 22 the distribution of stress in a semi-infinite, elastic, homogeneous, isotropic medium can be calculated by means of the Boussinesq equations. Terzaghi[1] and Jürgenson[2] amongst others, however, have produced tabulated solutions of the stress distribution due to the principal cases of loading met with in practice. From these tables it is possible to calculate the stress at any point or the distribution of stress on any horizontal, or vertical plane due to the following areas and systems of loading:—

(1) Circular area with uniform loading.
(2) Circular area with "triangular" loading.
(3) Long strip with uniform loading.
(4) Long strip with "triangular" loading.
(5) Long strip with "terrace" loading.
(6) Rectangular area with uniform loading.

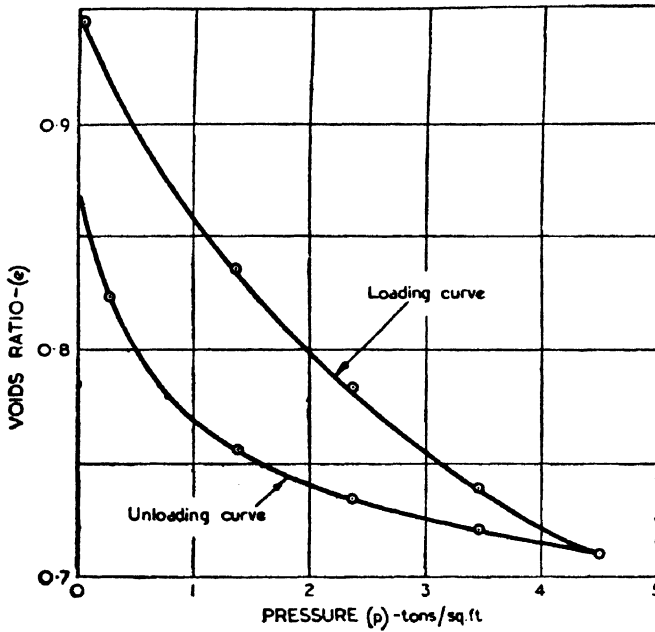



FIG. 23·4 TYPICAL VOIDS RATIO/EFFECTIVE PRESSURE RELATIONSHIP FOR AN UNDISTURBED SAMPLE OF LONDON CLAY

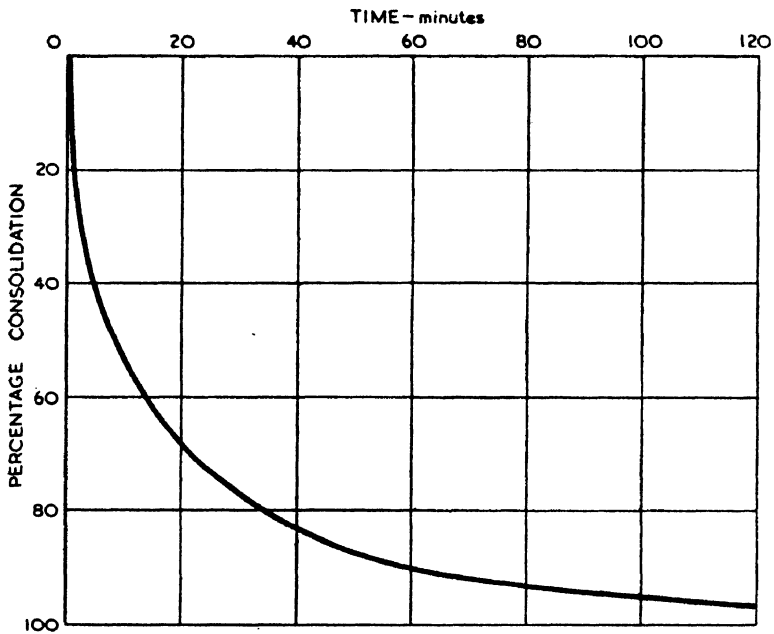



FIG. 23·5 PERCENTAGE CONSOLIDATION/TIME RELATIONSHIP FOR AN UNDISTURBED SAMPLE OF LONDON CLAY

23·31 "Triangular" loading is the case where the load varies uniformly from zero at the edges to a maximum at the centre, and "terrace" loading where the load varies uniformly from zero at one edge to a maximum value and then remains constant.

23·32 By a combination of some of the above types of loading it is usually possible to approximate very closely to the true distribution when dealing with other cases of loading and shapes of area.

23·33 In soil problems the assumption is made that the stress distribution is the same as that which would occur in the Boussinesq medium (a semi-infinite, homogeneous, isotropic, elastic medium).

23·34 Besides the determination of the stress distribution based on the Boussinesq equations, simplified methods are often employed in practice which assume that the load is carried down with a 1:1 or a 2:1 spread and that the stress distribution is uniform on any given horizontal plane. Fig. 23·6 shows a comparison of the Boussinesq curves for the variation with depth of the vertical stress under the centre of a circular area loaded uniformly, based on the Boussinesq equation and a 1:1 and 2:1 spread. A comparison of the stress distribution along a horizontal plane at a depth equal to the diameter of the loaded area is also given in Fig. 23·6. These comparisons show that the 1:1 spread under-estimates the stress as compared with the Boussinesq equation, the 2:1 spread being much closer. This suggests that for approximate calculations it is preferable to use the 2:1 instead of the 1:1 spread.

Estimation of Total Settlement

23·35 Whatever form of stress distribution is finally adopted, the procedure for calculating the total settlement of a structure is the same. Briefly the method is as follows:—

23·36 The effective stress in the compressible layer due to the overburden soil is calculated. The increase of stress in the soil due to the dead load of the structure is then estimated. The corresponding change in voids ratio of the compressible layer is found from the voids ratio/pressure relationship obtained from the laboratory consolidation test. From a knowledge of this change in voids ratio and the initial thickness of the compressible layer, the change in thickness can be calculated and hence the settlement of the structure.

23·37 It can be shown that the change in thickness, ρ_1, of a saturated soil layer due to a change in voids ratio is given by:—

$$\rho_1 = \left(\frac{e_1 - e_2}{1 + e_1}\right) H$$

where e_1 = initial voids ratio of soil
e_2 = final voids ratio of soil
H = thickness of soil layer.

23·38 In estimating the initial voids ratio existing in the compressible strata due to the overburden, the effective pressure must be used (see assumption (8) of Theory of Consolidation); that is, the pore-water pressure must be deducted from the total overburden pressure.

23·39 When determining the increase in stress in the compressible layer, the change in stress midway in the thickness of the layer is normally considered. This assumes that the distribution of stress through the layer is linear. In the case of thick layers, this may introduce appreciable errors and the actual distribution of stress through the layer should be used.

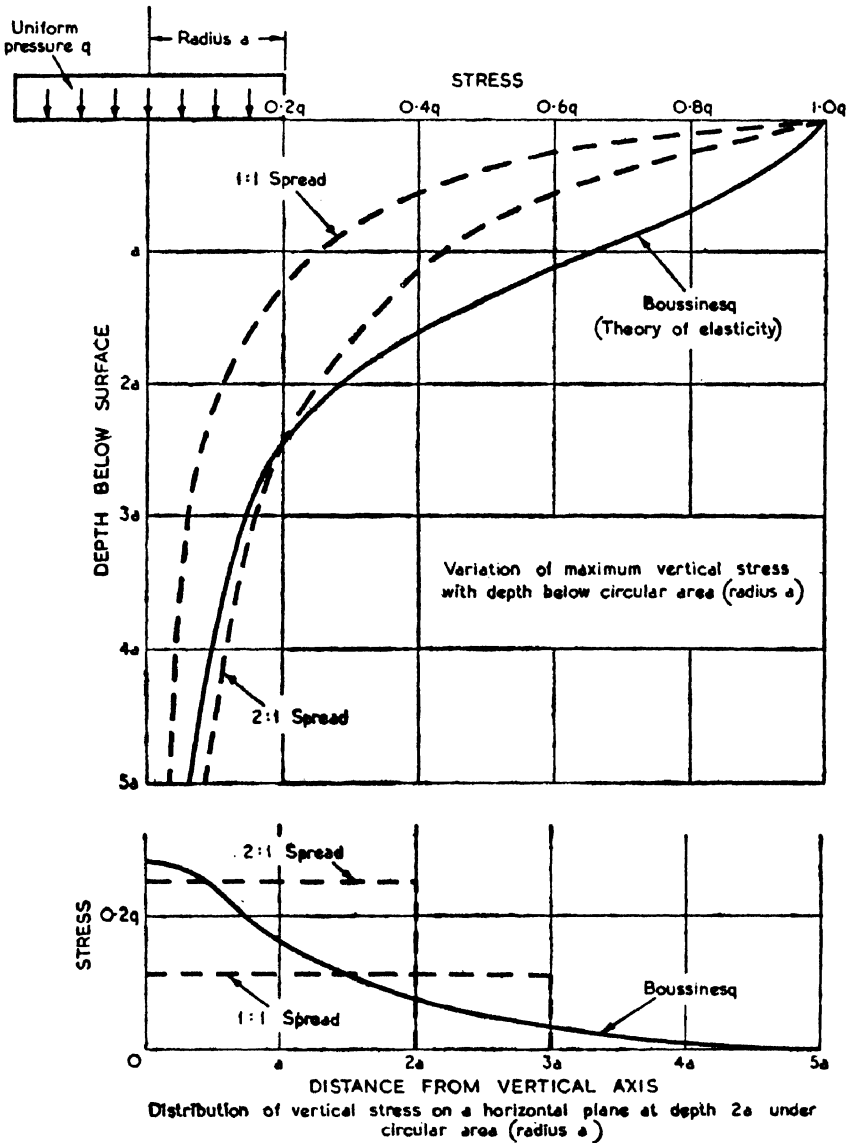



FIG. 23·6 COMPARISON OF THE STRESS DISTRIBUTION UNDER A UNIFORMLY LOADED CIRCULAR AREA

Calculated on the basis of the Boussinesq equation and 1:1 and 2:1 spread

23·40 The distribution of stress on any horizontal plane is not uniform and except in rough calculations the settlement under various points of the structure should be examined so that the distribution of settlement can be obtained.

Estimation of Settlement Rate

23·41 The rate of settlement of a proposed structure is found by using the percentage consolidation/time relationship obtained from the laboratory test. This graph is in reality the settlement/time curve for any structure but plotted to different scales. The 100-per cent consolidation corresponds to the total settlement of the structure calculated as indicated above, whilst the time scale is found from the fact that the rate of consolidation is inversely proportional to the square of the length of the drainage path. Any time t, say, on the time scale corresponds to a time

$$t \times \left[\frac{\text{length of drainage path in the field}}{\text{length of drainage path in the laboratory test}}\right]^2$$

for the proposed structure. This enables the settlement rate to be found.

23·42 Where the compressible layer underlying the structure extends to a considerable depth, the length of drainage path can be assumed to be equal to the thickness of that part of the layer carrying a measurable stress.

23·43 The assumption was made in the mathematical theory of consolidation that the total surcharge responsible for the consolidation is instantaneously applied in full. In practice, however, the rate of application of the consolidating load will depend upon the rate of construction of the structure. In addition, the building of most structures, except perhaps embankments, requires the excavation of at least a few feet of the foundation soil and thus initially there will be a relief in the overburden pressure and there will be a tendency for the underlying compressible strata to swell. Settlement below the initial datum will not commence until the building loads exceed the weight of excavated foundation soil. Thus, if an accurate prediction is required for the movements of a structure both during and after construction, the settlement/time curve has to be corrected to allow for the slow application of load. The total settlements, however, after a long period of time will not be affected by the initial rate of loading.

23·44 Terzaghi has evolved a method for calculating the rate of settlement when the load is applied slowly as during construction. This method is based on the assumption that if the load is applied at a uniform rate, the settlement that would occur at the end of the construction period, would be equal to the settlement that would have occurred at half the time, if the entire construction load had been applied instantaneously at the beginning. Similarly, the settlement at any time during the construction period is equal to that occurring at half that time for instantaneous loading, but since the load acting is not in fact the total load, the settlement must be reduced in the proportion of that load to the total load. The settlement/time curve for the period subsequent to the completion of the structure will follow the portion of the instantaneous loading curve from the time equal to half the construction period.

23·45 A simple graphical procedure based on the above assumption for estimating the settlement rate when the loading rate is uniform is shown in Fig. 23·7.

OAB is the load/time curve.

OCD is the settlement/time curve if the total load is applied instantaneously at the beginning.

t_c is the construction period.

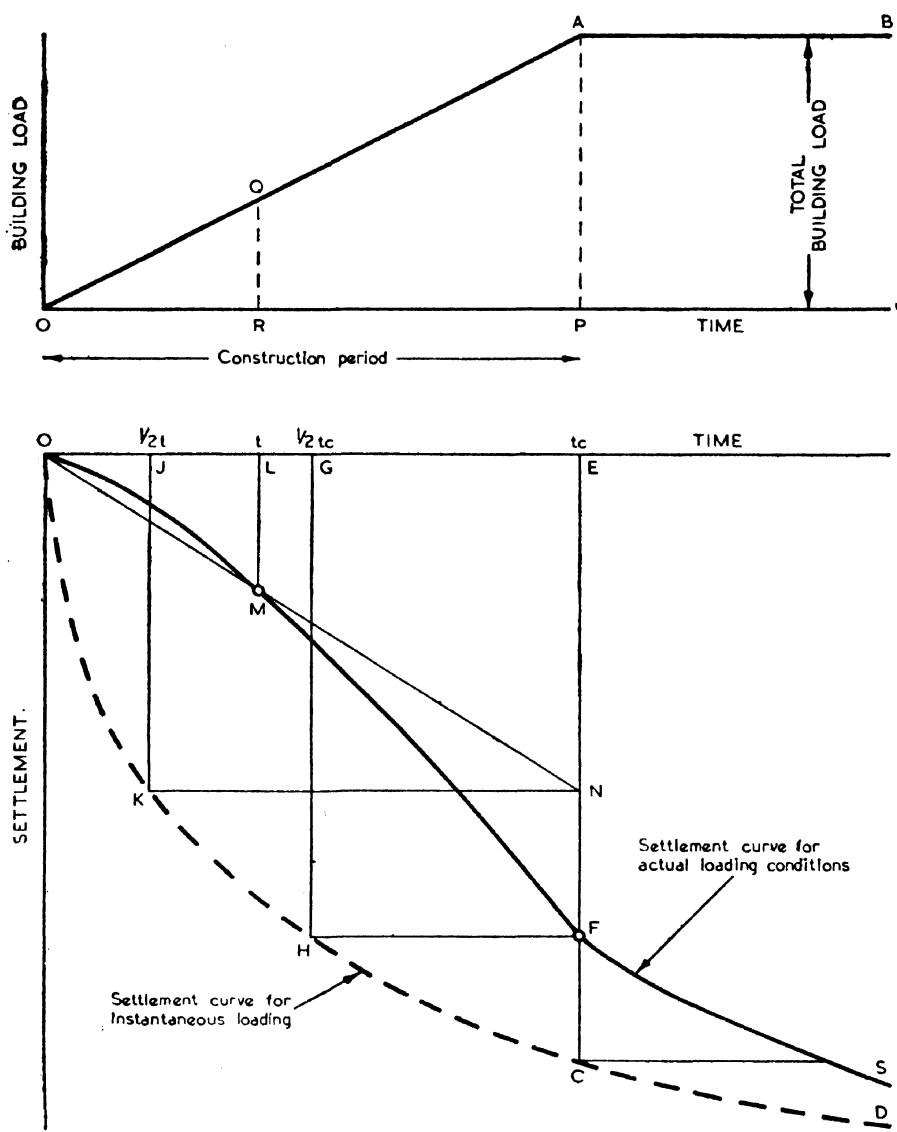



FIG. 23·7 GRAPHICAL CONSTRUCTION FOR ESTIMATING SETTLEMENT WHEN LOAD IS APPLIED AT A UNIFORM RATE

At the end of the construction period t_c the settlement is equal to that occurring at $\frac{1}{2}$ t_c if all the load were applied at zero time,

$$\text{i.e. settlement} = GH = EF$$

At any time t during construction the settlement will be equal to that occurring at $\frac{1}{2}$t for instantaneous loading, reduced in the proportion of the load acting at time t to the total load at t_c

$$\text{i.e. settlement at t} = JK \times \frac{QR}{AP}$$

$$= JK \times \frac{LM}{EN} \text{ since } \frac{LM}{EN} = \frac{QR}{AP}$$

$$= JK \times \frac{LM}{JK} \text{ since } JK = EN$$

$$= LM.$$

Thus the procedure to find the settlement at any time t is to project the settlement at $\frac{1}{2}$t under instantaneous loading conditions on to the ordinate at the time t_c, the end of the construction period. The intersection of the line from 0 to this ordinate with the ordinate at t gives the settlement at t.

23·46 The settlement curve after the end of the construction period, FS, follows the portion of the instantaneous loading curve from the point H.

23·47 An alternative method of determining the settlement/time curve when the load is gradually applied and especially when the rate of loading is not uniform is to divide the loading/time curve into a series of suddenly applied increments. A series of settlement/time curves are plotted for the different increments of loading on a common system of axes, the origins of the different curves being displaced along the time axis to their correct positions. The ordinates are then added to produce a single curve.

23·48 The accuracy with which the predicted curves of settlement have been known to follow the actual settlements is sometimes very good. For example, the actual and predicted settlements of the foundation of Clendening Dam[3] in the U.S.A. agreed to within 5 per cent throughout the movement of 2·6 ft.

23·49 An example of the method of settlement analysis in the case of a road embankment is included in the appendix to this chapter.

SOME PRACTICAL ASPECTS OF CONSOLIDATION

23·50 If a laboratory consolidation test is carried out on a remoulded instead of an undisturbed sample of soil, the gradient of the voids ratio/effective pressure curve is very often much steeper in the case of the remoulded sample. This means that a structure founded on remoulded or disturbed soil may settle more than on the same soil if it were undisturbed.

23·51 One of the methods frequently employed for determining the "safe load" for the foundations of structures so that the settlements will be negligible is to carry out loading tests on small areas of the ground. With small-scale

loading tests, however, the stresses due to the load are confined to the top few feet of ground and therefore the results of such tests can be most misleading. Small-scale tests and even large-scale tests do not take into account the time factor which is responsible for the continuing settlement of a structure many years after its construction. Except where non-compressible soil exists to a considerable depth (at least 1½ times the largest dimension of the structure) small-scale loading tests on the site are likely to be of little value.

23·52 With most structures it is desirable to reduce settlements to a negligible amount or so increase the rate of settlement that the major portion of the movement has occurred before the constructional work is complete. Two methods of reducing the amount of settlement to any desired value are:—

(1) Reducing the load on the soil by increasing the area of the foundation. This might necessitate the use of a concrete raft instead of a footing foundation.

(2) Excavating to such a depth that the load of the removed overburden approaches that of the dead load of the structure.

23·53 The rate of settlement of a structure can be increased by reducing the length of the drainage path along which the soil water has to travel to reach the drainage layer as it is squeezed out of the soil. One way by which this can be effected is by the installation of vertical sand drains. These consist of vertical holes back-filled with sand or filter material which are bored through the soil layers liable to consolidate. The drains are installed at about 6- to

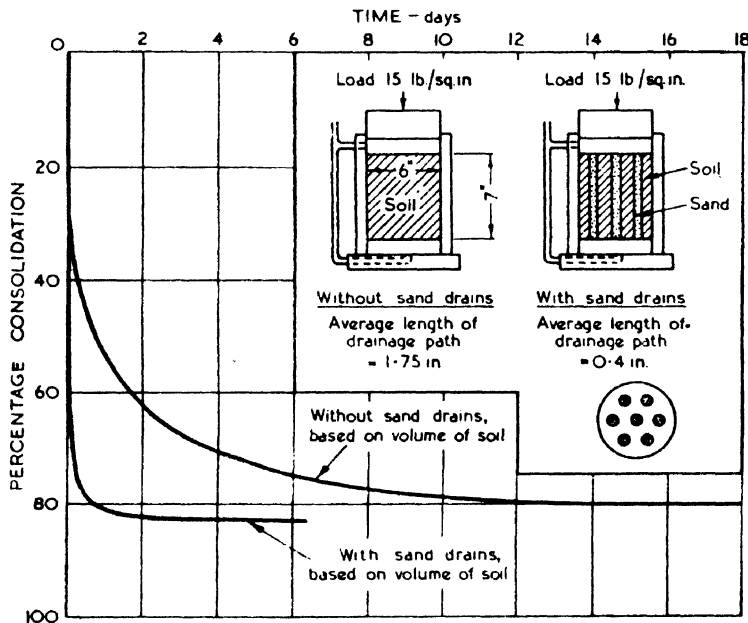



FIG. 23·8 LABORATORY INVESTIGATION INTO THE EFFECT OF VERTICAL SAND DRAINS

(After Porter)

10-ft centres in all directions and are generally about 9 to 12 in. in diameter. Holes of 18-in. diameter have been employed however, but at a greater distance apart—about 12 ft. The holes can be made by boring or jetting depending upon the soil and site conditions. In Sweden, vertical tubes constructed of cardboard have been employed instead of sand columns.

23·54 The results of a laboratory investigation[4] into the effect of vertical sand drains on the rate of consolidation is shown in Fig. 23·8. It will be seen that, on the basis of an equal volume of compressible soil, the effect of the sand drains was to increase the rate of consolidation so that 80 per cent of the settlement was completed within $\frac{3}{8}$ of a day compared with 16 days for the specimen containing no sand drains.

APPENDIX TO CHAPTER 23

Illustration of the Method of Settlement Analysis in the case of a Road Embankment

23·55 It is proposed to calculate the distribution of settlement of the foundation of an embankment 25 ft high, composed of well compacted granular material which is underlain by a layer of London clay 20 ft thick. The dimensions of the embankment and the thicknesses and positions of the various soil strata are shown in Fig. 23·9. The bulk densities of the soil strata and the embankment material are also given in Fig. 23·9. The water-table is assumed to be at the surface of the ground.

23·56 As the embankment is composed of well compacted granular material, the settlements occurring within the embankment itself are assumed to be so small in comparison with the compression of the clay layer that they can be neglected in the analysis.

23·57 For the sake of simplicity, the average vertical stress in the compressible clay layer in any longitudinal vertical plane is assumed to be equal to the stress at the mid-point in the layer, i.e. the stress at a depth of 12·5 ft (see Fig. 23·9).

The effective stress at a depth of 12·5 ft before the embankment is constructed,

$$\sigma_1 = \frac{2{\cdot}5(120 - 62{\cdot}4) + 10(125 - 62{\cdot}4)}{2240} \text{ tons/sq.ft}$$

$$= \frac{2{\cdot}5 \times 57{\cdot}6 + 10 \times 62{\cdot}6}{2240} \text{ tons/sq.ft}$$

$$= 0{\cdot}344 \text{ tons/sq.ft.}$$

23·58 The distribution of stress at a depth of 12·5 ft due to the embankment load is calculated using Jürgenson's tabulated solution[8] for a strip area with triangular loading. The cross-section of the embankment and therefore the loading diagram can be considered as the difference between a large triangle of base 100 ft and a similar smaller triangle of base 50 ft. The stresses caused by the embankment are found by deducting the stresses due to the smaller triangle from those due to the larger triangle.

23·59 Fig. 23·9 shows the calculated distribution of vertical stress along a horizontal plane at a depth of 12·5 ft due to the embankment load.

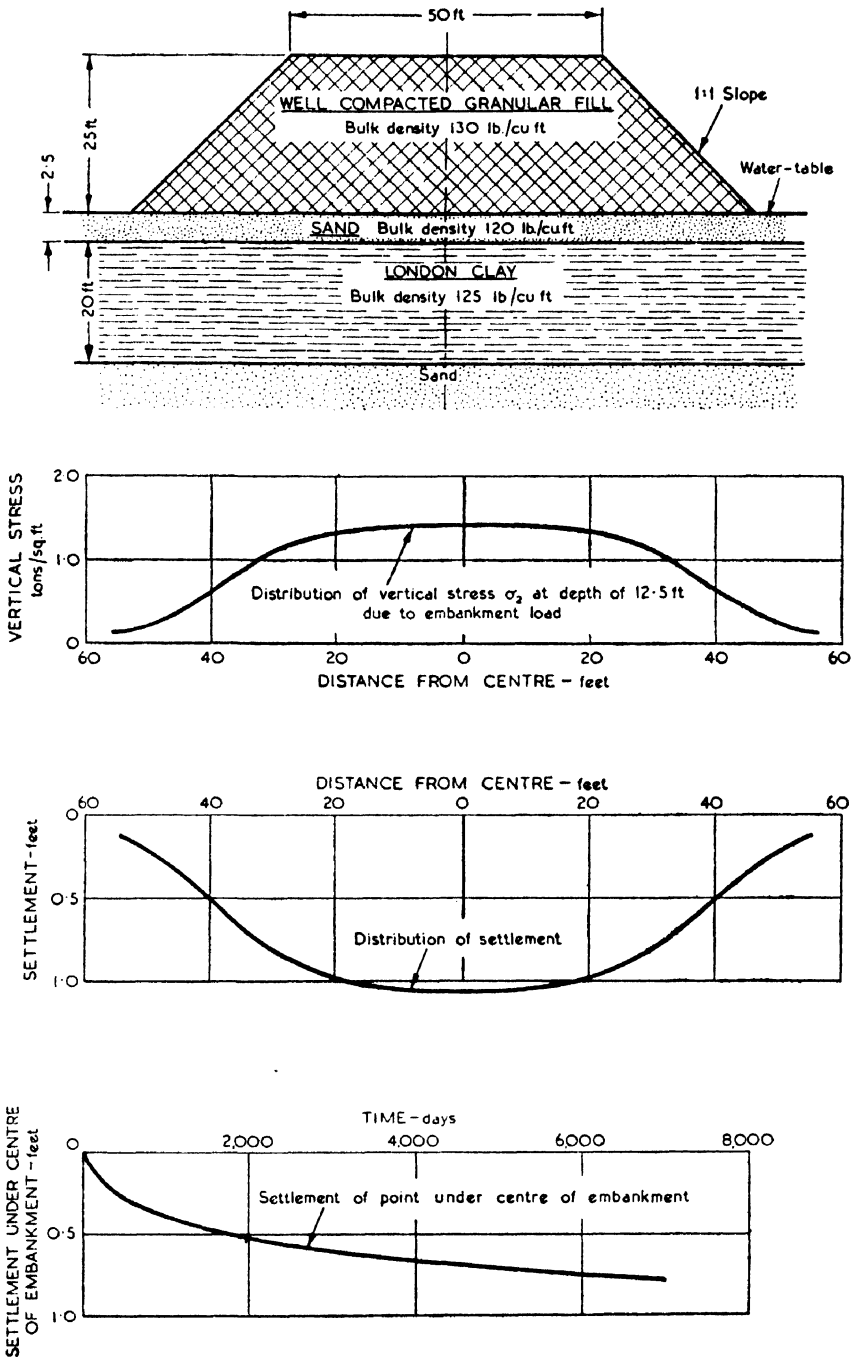



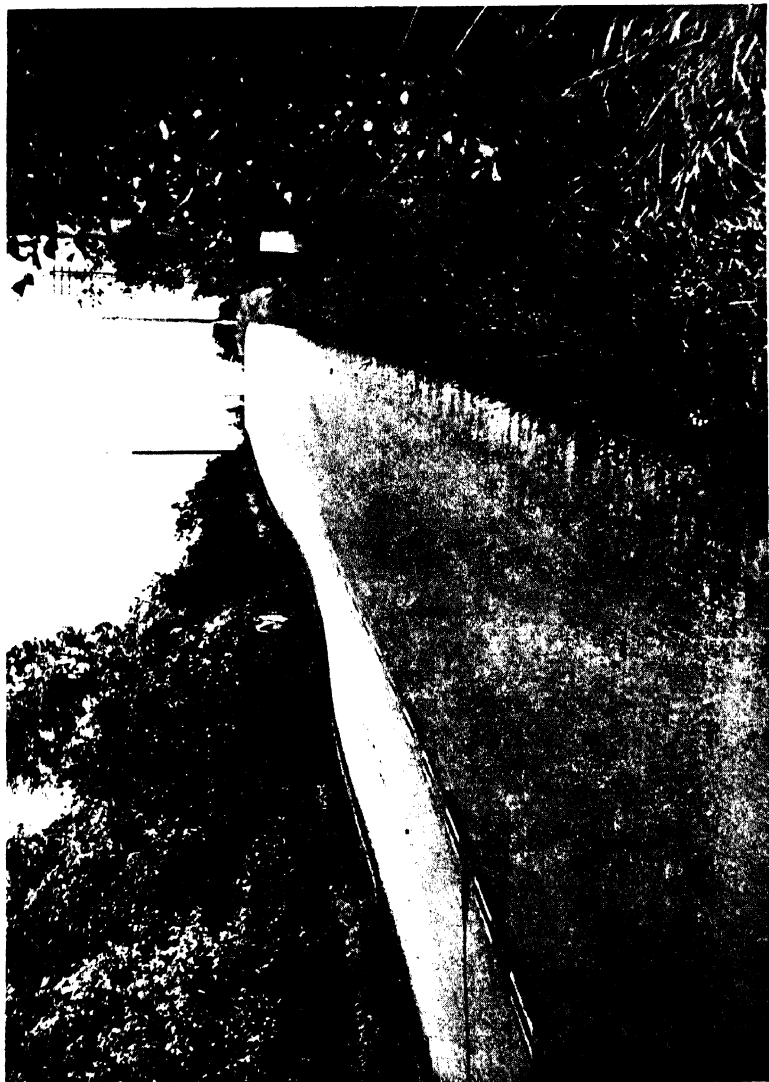
FIG. 23·9 CROSS-SECTION OF ROAD EMBANKMENT WITH STRESS AND SETTLEMENT CURVES

WAVINESS OF CONCRETE SHOWN IN A ROAD CONSTRUCTED ON A PEAT SUBGRADE

PLATE 23·1

DIFFERENTIAL SETTLEMENT DUE TO CULVERT UNDER A ROAD FOUNDED ON PEAT

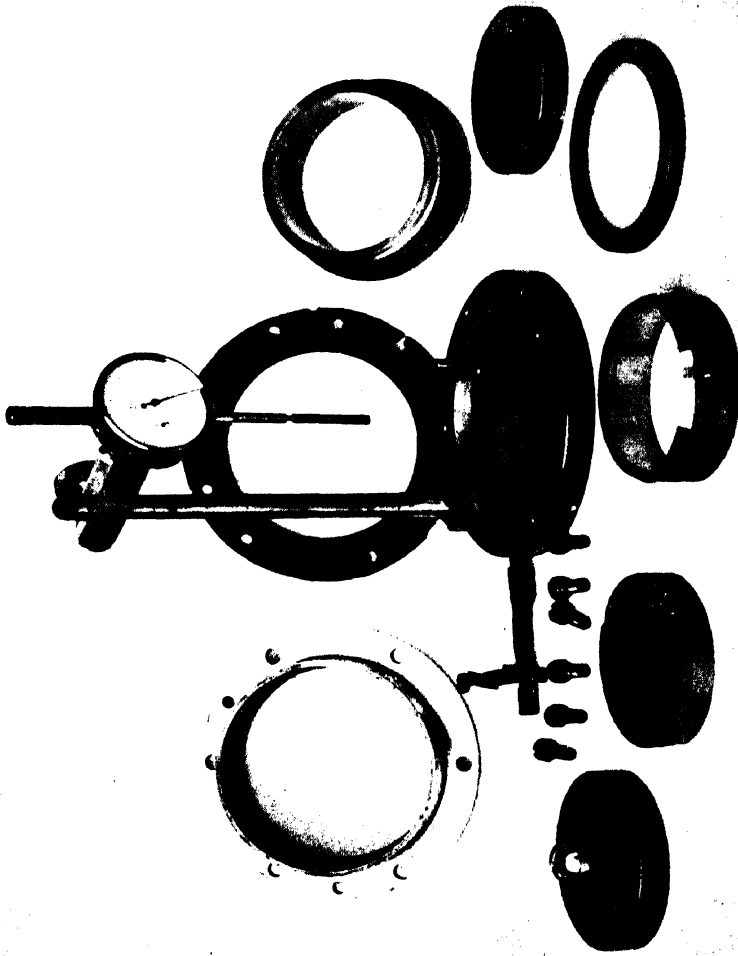
PLATE 23·2

DISMANTLED CONSOLIDATION APPARATUS
used at the Road Research Laboratory

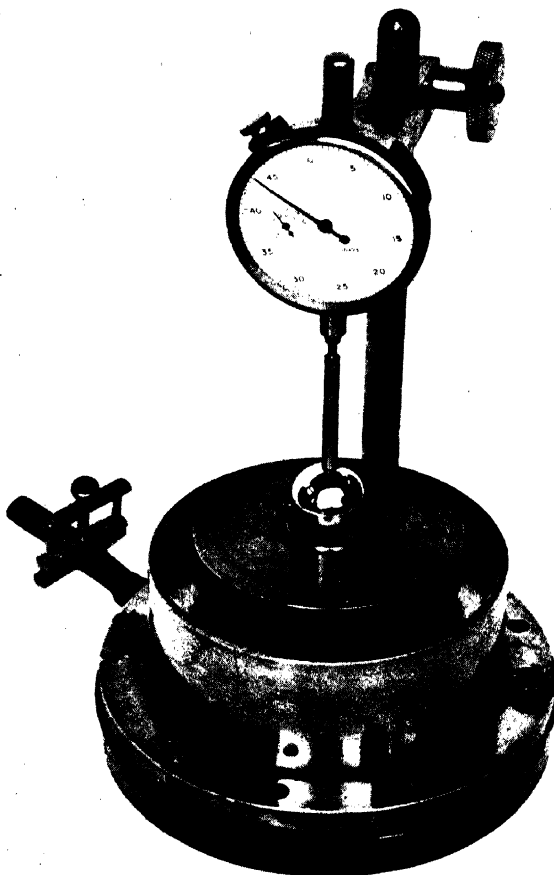
PLATE 23·3

ASSEMBLED CONSOLIDATION APPARATUS
used at the Road Research Laboratory

PLATE 23·4

23·60 The transverse distribution of settlement under the embankment is found by calculating the settlement at various points under the embankment. For the sake of brevity, only the detailed calculations of the settlement of a point under the centre of the embankment will be given. The settlement for other points is found in exactly the same way.

Effective stress before embankment construction σ_1 = 0·344 tons/sq. ft

Referring to Fig. 23·4, the corresponding voids ratio in the clay e_1 = 0·910

Additional stress due to the embankment σ_2 = 1·423 tons/sq.ft (See Fig. 23·9.)

∴ Total effective stress after embankment construction σ_3 = 1·423 + 0·344 = 1·767 tons/sq.ft

Corresponding voids ratio, e_2 = 0·810

Settlement or change in thickness of the clay layer

$$= \left(\frac{e_1 - e_2}{1 + e_1}\right) H$$

$$= \left(\frac{0{\cdot}910 - 0{\cdot}810}{1 + 0{\cdot}910}\right) 20 \text{ ft}$$

$$= \frac{0{\cdot}1 \times 20}{1{\cdot}91} \text{ ft}$$

$$= 1{\cdot}05 \text{ ft.}$$

23·61 Similarly, the settlement at various points under the embankment can be calculated. It is convenient to use a tabulated method (Table 23·1).

TABLE 23·1

CALCULATION OF THE VARIATION OF SETTLEMENT UNDER THE EMBANKMENT

	Distance from centre line of embankment - ft				
	0	12·5	25	37·5	50
σ_1	0·344	0·344	0·344	0·344	0·344
σ_2	1·423	1·403	1·233	0·724	0·210
$\sigma_3=\sigma_1+\sigma_2$	1·767	1·747	1·577	1·068	0·554
e_1	0·910	0·910	0·910	0·910	0·910
e_2	0·810	0·812	0·822	0·855	0·890
e_1-e_2	0·100	0·098	0·088	0·055	0·020
$1 + e_1$	1·910	1·910	1·910	1·910	1·910
Settlement (ft) $= \left(\frac{e_1-e_2}{1+e_1}\right) H$	1·05	1·03	0·92	0·58	0·21

σ_1, σ_2, σ_3 are in tons/sq.ft

Calculation of the Rate of Settlement

23·62 The length of the drainage path in the field is equal to half the thickness of the clay layer as permeable sand exists on top and below the clay,

i.e., The length of the drainage path in field = 10 ft

The length of the drainage path in the laboratory test = 0·2 in.

$$\therefore \left(\frac{\text{length of drainage path in field}}{\text{length of drainage path in laboratory test}}\right)^2$$

$$= \left(\frac{10 \times 12}{0 \cdot 2}\right)^2 = 360{,}000$$

23·63 Therefore, the corresponding percentage consolidation in the field as compared with the laboratory test occurs after a period 360,000 times as long.

e.g., 50 per cent consolidation requires 8 min. in the laboratory

∴ 50 per cent consolidation in the field will require:—

$$\frac{8 \times 360{,}000}{60 \times 24} = 2{,}000 \text{ days.}$$

23·64 The settlement corresponding to 50-per cent consolidation is found from the total settlement graph and in this way the time relationship can be plotted. The settlement/time curve for a point under the centre of the embankment is given in Fig. 23·9.

SUMMARY

23·65 The large settlements occurring with structures founded on compressible soils are due almost entirely to consolidation of the underlying strata. Consolidation is the process whereby the voids of a saturated soil are reduced, generally by the action of static loading acting over a long period of time, the rate of consolidation depending upon the permeability of the compressible strata.

23·66 Terzaghi has evolved a mathematical theory of consolidation based upon certain simplifying assumptions, such that the total movement and rate of settlement of a proposed structure can be predicted from the results of laboratory tests made on undisturbed samples of the underlying soil. This mathematical theory is explained briefly and the laboratory tests described. The method of settlement analysis is given in some detail together with the modifications necessary for the more usual case when the total load is not instantaneously applied.

23·67 A brief survey is made of some of the more practical aspects of consolidation including the methods by which settlement can be reduced or the rate of settlement so increased that most of the movement takes place before the constructional work is complete.

23·68 An example of the method of settlement analysis in the case of a road embankment is included in an appendix.

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CHAPTER 24

SETTLEMENT OF EMBANKMENTS

INTRODUCTION

24·1 The settlement of a road surface constructed on an embankment usually results from:—

(1) The reduction in the voids in the fill material of the embankment produced by the weight of the layers of fill above, by the effect of traffic loads and possibly by the action of weather.

(2) The settlement of the foundation upon which the embankment is built. This arises from the elastic compression of the foundation due to the superimposed load and from consolidation in the case of saturated compressible soils, e.g. clays.

24·2 The method of analysis for determining the amount and rate of settlement due to consolidation is described in Chapter 23. Settlements due to the elastic compression of the foundation are very small and except in the case of high embankments can usually be neglected.

24·3 Where embankments consist of clay compacted at a moisture content close to the plastic limit (the usual condition of clays in this country), the soil will contain only a very small amount of air voids and can be treated as a saturated soil, the consolidation theory being used to estimate the settlement. If, however, the fill material contains an appreciable amount of air voids the settlement cannot be determined in this way and at present there is no proved laboratory method of estimating the probable increase in density of an unsaturated fill under load. Estimates of the settlements of embankments in these circumstances must therefore be based on information derived from measurements made on actual embankments. Some useful information was obtained by the Laboratory from full-scale experiments carried out during 1938 to 1948 to determine the settlement of chalk and clay embankments. A few similar investigations have also been made in the U.S.A. The information is summarized in this chapter to afford a guide to those interested in the problem.

EFFECT OF COMPACTION ON SETTLEMENT

24·4 The state of compaction obtained at the time of the construction of an embankment will affect the extent of the ensuing settlement. This is shown by the results of measurements made during and subsequent to the construction of a number of embankments in Ohio[1]: these results have been analysed to give Fig. 24·1 which shows the relation between the settlement of the embankments 3 years after construction and the relative compaction obtained at the time of construction. The embankments ranged in height from 7 to 15 ft, and the fill material consisted of a silty clay, the average liquid limit being 37 per cent and plastic limit 21 per cent. The average optimum moisture content of the

fill was 18 per cent, the corresponding average maximum dry density being 108 lb./cu.ft. It will be seen from the figure that the settlement decreased markedly with the increased compaction of the fill.
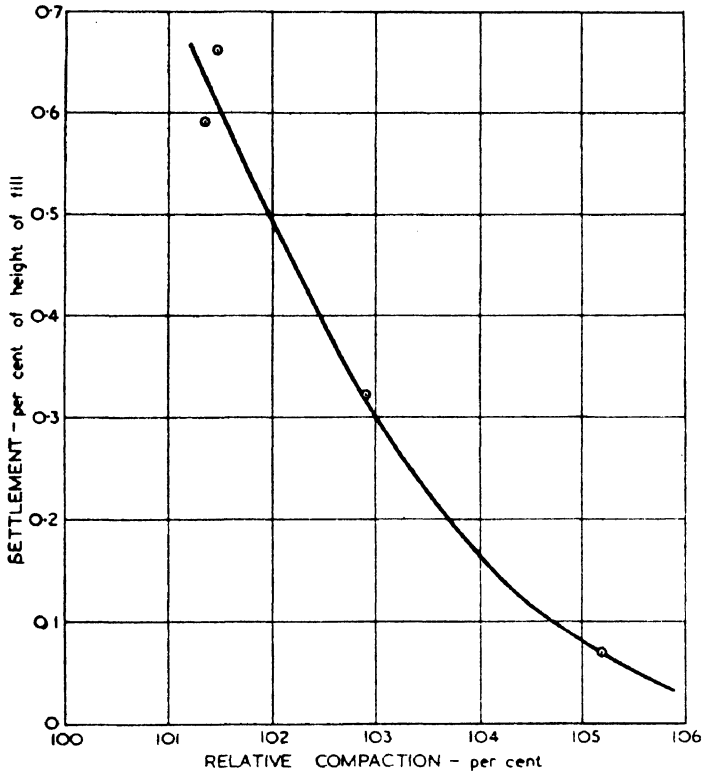
FIG. 24·1 RELATIONSHIP BETWEEN THE PERCENTAGE SETTLEMENT OF EMBANKMENTS AND THE RELATIVE COMPACTION OF THE FILL AT THE TIME OF CONSTRUCTION

Silty clay embankments in Ohio, U.S.A., 3 years after construction

24·5 Investigations of the settlement of a chalk embankment carried out by the Laboratory showed that where the dry density of the chalk fill was approximately 80 lb./cu.ft the settlement after 1 year was about 2·5 per cent, but where the dry density was 88 lb./cu.ft the settlement was negligible.

24·6 The following broad conclusions were drawn from earlier field experiments carried out by the Laboratory[2][3].

(1) On lightly compacted chalk embankments the recorded settlements after 5 to 7 years were from 0·4 per cent to 1·1 per cent of the height of fill, the corresponding value for better compacted chalk embankments amounting to 0·2 per cent.

(2) The recorded settlements after 5 to 7 years of lightly compacted clay embankments were from 1·1 to 1·9 per cent of the height of fill and of better compacted ones about 0·2 per cent.

EFFECT OF EMBANKMENT HEIGHT ON SETTLEMENT

24·7 The results of field investigations[8] have indicated that the settlement is roughly proportional to the height of the embankment. This means that where the depth of fill is reduced by the presence of some structure, such as an underbridge for example, differential movements will take place. This is illustrated in the case of the embankment shown in Fig. 24·2. An arch underbridge through the embankment had the effect of causing considerable differential settlements, the movement of the road over the underbridge being about 5 in. compared with about 10 in. for the rest of the embankment (see Fig. 24·2). A photograph of the road along this portion of the embankment (Plate 24·1) shows the effect of these differential movements on the appearance of the road.

RELATIONSHIP BETWEEN SETTLEMENT AND TIME

24·8 A knowledge of the settlement/time relationship for an embankment is important for it is only the movement occurring subsequent to the construction of the road which will affect the surface levels. Often a year or more is allowed to elapse between the completion of the earthworks and the commencement of the road construction, in which case most of the settlement has probably occurred by the end of this period.

24·9 Observations on embankments have shown that the settlement is usually rapid immediately after construction and that the rate decreases with time. Fig. 24·3 shows the settlement/time curves obtained with a chalk embankment 45 ft high. About 75 per cent of the total settlement occurring in 7 years took place during the first 2 years.

24·10 In the case of saturated clay fills or those containing a negligible amount of air voids an estimate of the rate of settlement can be made from the results of laboratory consolidation tests. The rate of settlement in the field will depend on the distance water has to travel to reach a drainage layer as it is squeezed out of the soil. In several embankments recently constructed in the U.S.A., the length of the drainage path was reduced and the rate of settlement increased by the installation of sand drainage columns in the fill.

24·11 With unsaturated and granular soils the rate of settlement will depend on the nature of the material and probably on the height and degree of compaction of the fill but no precise information is available. It is possible that a fairly close estimate could be made from the results of laboratory consolidation tests similar to those which are used for saturated soils.

POSSIBLE REMEDIAL MEASURES TO OVERCOME DIFFERENTIAL SETTLEMENTS

24·12 On either side of bridges at the junctions of the abutments and the approach embankments, serious differential settlement of the road surface often occurs as the result of the poor compaction of the fill or the consolidation
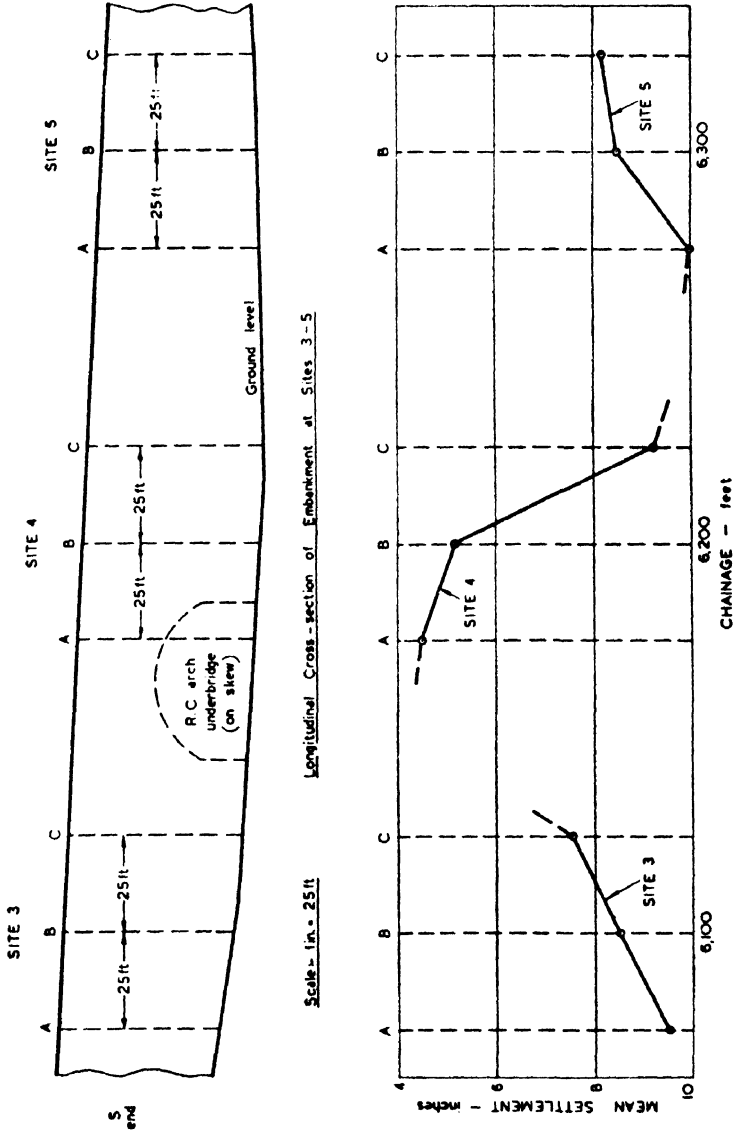
FIG. 24·2 LONGITUDINAL SECTION AND DISTRIBUTION OF SETTLEMENT OF A CHALK EMBANKMENT 45 FT HIGH
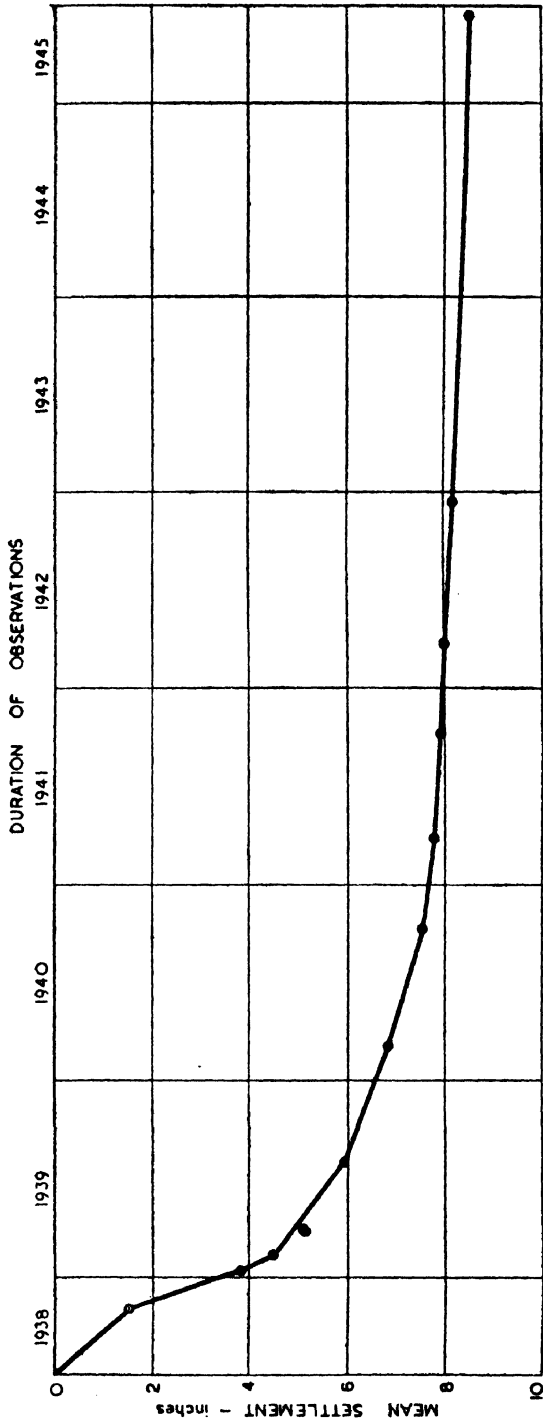
FIG. 24·3 TYPICAL SETTLEMENT/TIME CURVE FOR A CHALK EMBANKMENT 45 FT HIGH

ROAD SURFACE SHOWING UNDESIRABLE SETTLEMENTS ON EITHER SIDE OF AN UNDERBRIDGE

PLATE 24·1

(A) SETTLEMENT OF CONCRETE SLABS ON EMBANKMENT ADJACENT TO A BRIDGE ABUTMENT
(Note: The sunken surface has been partly made good with a bituminous material)

(B) SUNKEN CONCRETE SLABS AFTER BEING RAISED LEVEL BY PRESSURE-GROUTING PROCESS

PLATE 24·2

of the foundation soil. Provision is often made for this by interposing a section of flexible pavement (on the German motor roads, granite setts were used) on either side of the road bridges. Any undesirable settlements can then be made good with the minimum of inconvenience.

24·13 Settlements of concrete road slabs have been remedied successfully by the application of "mud-jacking" or pressure grouting. This process consists in raising the road to its correct level by pumping a cement grout under the sunken slabs through holes bored through the concrete. Pressures of the order of 40 to 80 lb./sq.in. are usually necessary. The average cost of pressure grouting work carried out in 1945 by one authority amounted to about 11s./sq. yd of road surface.

24·14 Plate 24·2 shows the approaches to a bridge before and after raising the slabs by pressure grouting.

SUMMARY

24·15 The settlement of embankments is reviewed in the light of experience gained during the studies made by the Road Research Laboratory from 1938 to 1948 of the settlement of several chalk and clay embankments. Data obtained from investigations in the U.S.A. are also included. The effects of compaction and height of fill on the settlement are discussed and consideration is given to the settlement/time relationship.

24·16 The settlement of concrete road slabs has been successfully remedied by means of pressure grouting ("mud-jacking").

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CHAPTER 25

CONSTRUCTION OF ROADS ON SWAMPY GROUNDS

INTRODUCTION

25·1 Where roads have to be constructed over unstable ground composed of peat or soft clays it is essential to use methods of construction which will ensure either the removal of the soft material or its complete consolidation before the structure is completed. This chapter describes methods of removing the soft material; a method of accelerating the consolidation process is described in Chapter 23. Although this chapter refers specifically to the removal of peat, the methods are equally applicable to soft clays and other similar materials.

EXCAVATION AND DISPLACEMENT

Total Excavation

25·2 Where the depth of peat is small, less than 10 ft, it is usual to remove it completely with excavators (Fig. 25·1). Starting work at one edge, excavators are usually able to work on the solid bottom under the soft ground,

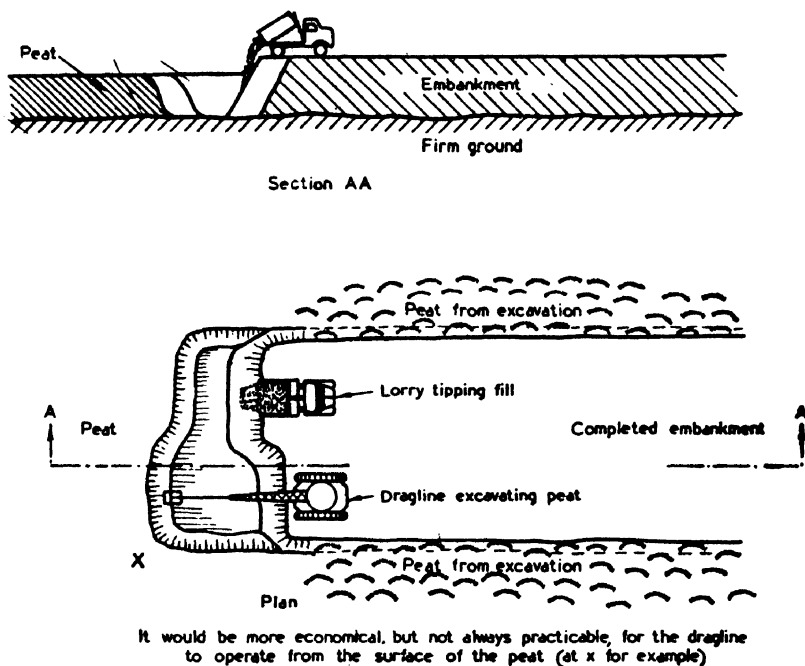



FIG. 25·1 METHOD OF COMPLETE EXCAVATION OF PEAT

unless the water level is too high. The filling is kept close behind the excavation in order to reduce the possibility of the sides of the excavation slipping in again. The excavated material can then be dumped on the side slopes of the embankment. Where an old road is being widened, excavation should be taken down to a sound foundation on either side, but any peat underneath the existing road may be allowed to remain, since it will be surrounded by better material.

Partial Excavation and Partial Displacement

25·3 In some cases where the peat is over 10 ft deep, and particularly where it is very fibrous or contains much wood near the surface and is soft below, it is excavated to a depth of about 8 or 10 ft and the lower part displaced by the superimposed weight of soil in the embankment (Fig. 25·2). It is usual to put a surcharge load of about 5 ft of fill above finished level to speed up the displacement of the peat: some of this may have to be removed but a considerable portion of it will become part of the finished embankment. If the lower part of the peat is too solid to flow under the weight of the embankment it may be necessary to use jetting as described below.

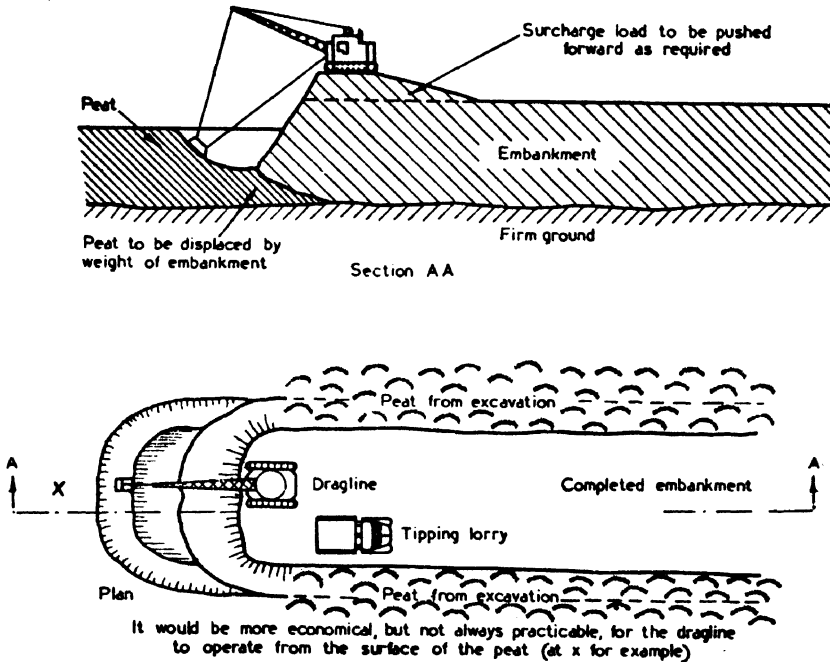



FIG. 25·2 METHOD OF PARTIAL EXCAVATION AND PARTIAL DISPLACEMENT OF PEAT

Settlement by Jetting

25·4 This method is used wherever practicable if the methods described above fail or can be seen to be unsuitable. It can be used economically only if the fill material through which the jets pass is free of stones or heavy clay. The underlying principle of the method is that when the strength of the ground

is sufficient to support the embankment, but low enough for there to be considerable settlement by consolidation, then by increasing its moisture content by jetting, the strength of the peat or clay can be reduced sufficiently to enable it to be displaced by the filling. Pressures of from about 30 to 250 lb./sq.in. are used with pipes of ¾- to 1-in. diameter spaced at a distance in each direction about equal to the thickness of the peat.

25·5 The jets are sunk rapidly to the bottom of the soft material and then slowly withdrawn. The cost of jetting on one large scheme was about sixpence per cubic yard of displaced material. Alternatively, jetting may be used to increase the weight and density of the fill material by saturating it so that it will then displace the peat or clay. The method used is similar except that lower pressures are required, 50 lb./sq.in. being about the maximum that is generally needed. The latter method has been used extensively in Michigan, U.S.A.

REMOVAL BY SUCTION PUMPS

25·6 In the U.S.A. soft ground is sometimes removed from below water level by suction pumps mounted either on land or on a boat, or by other types of dredger. The application of this method is limited and no estimate of cost is available.

DISPLACEMENT BY BLASTING

25·7 Except in open country, blasting methods are used only if the methods described above are unsuitable. In these methods anything up to half of the material displaced is moved by the weight of the fill, the remainder being moved by the explosion.

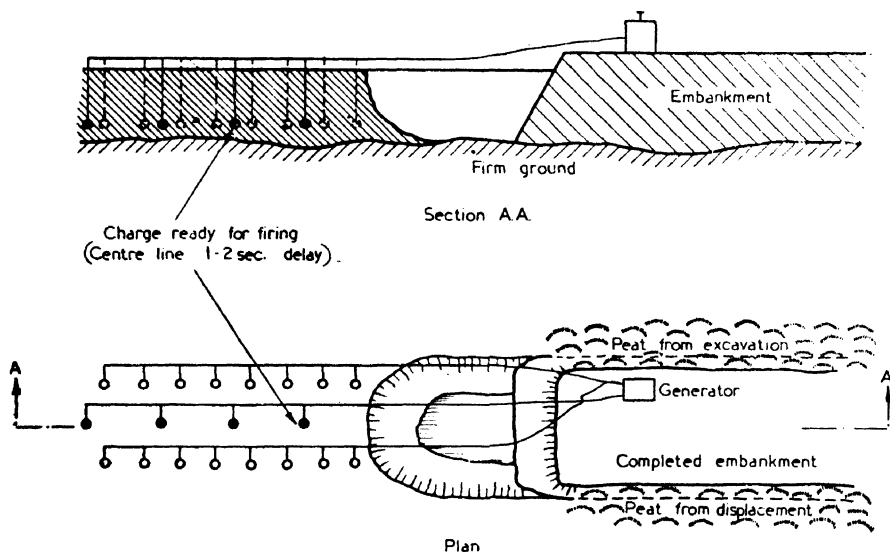



FIG. 25·3 TRENCH-SHOOTING METHOD OF BOG BLASTING

25·8 There are two principal methods of displacing the peat with fill material in which blasting is used, both of them having three main variants. The first method, trench shooting (and its variants), is used only for peat deposits at the surface and less than about 50 ft in depth. The second, or underfill method, can be used when a certain amount of soil overlies the peat, and has been used where the peat is over 70 ft thick. In all these methods the quantity of explosive required will vary with the nature of the peat, but will be between about ¼ and 3 lb. per cu.yd of peat to be displaced.

Trench-Shooting Method

25·9 This method can be used where the peat is not more than 20 ft deep and is fairly free from a tendency to slip. One or more rows of charges are placed near the bottom of the peat, usually by jetting, and fired, leaving an open trench in which the fill can be placed. The distance apart of the charges is usually about half to two-thirds of the thickness of the peat (Fig. 25·3). Where a considerable length of construction is to be carried out, the blasting is generally done in sections.

Toe-Shooting Method

25·10 This method, which is suitable for use in peat that is soft and liable to slip, is somewhat similar to the last method except that all the charges are close to the last placed part of the fill. The fill is carried forward until it forces the peat up in a wave ahead of it (Fig. 25·4). At this stage the charges are placed and fired, and the work is then continued until it is necessary to carry out further blasting.

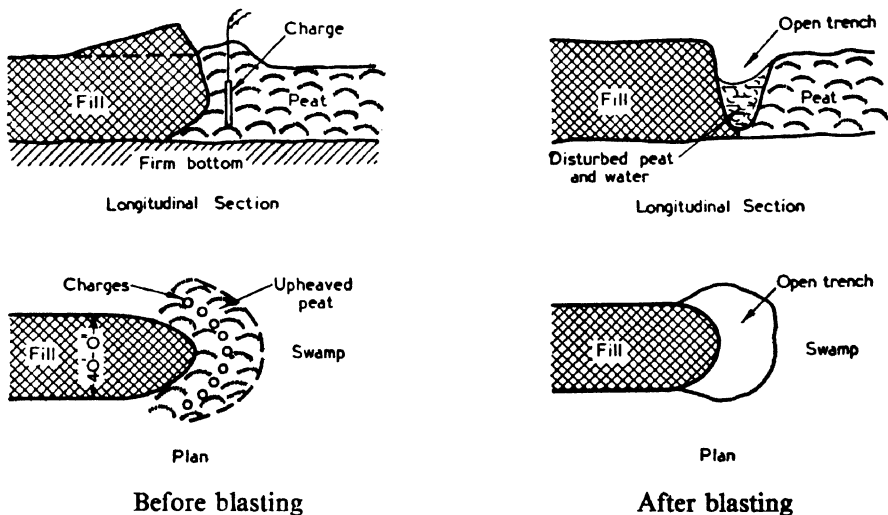



FIG. 25·4 TOE-SHOOTING METHOD OF BOG BLASTING

Torpedo Blasting Method

25·11 This method is essentially the same as the toe shooting method but it is used for thicker deposits and has been used for peat up to 50 ft thick.

The difference lies in the fact that instead of the whole of the charge being at the bottom of the peat, the sticks of explosive are tied at various points to a post about 10 ft long. Several of these torpedoes are placed upright in the upheaved peat close to the end of the fill. The distance between torpedoes is usually about 5 to 10 ft in each direction.

Underfill Methods

25·12 Underfill methods are used in cases where some sound material overlies the peat or where the top of the peat is very fibrous and matted, and also where a very great thickness of peat has to be dealt with. The amount of preliminary work on site investigation is much greater than for other methods and considerable experience is needed in estimating the size and spacing of the charges. The three variants of the method and their application are as follows:—

(1) Construction of the full width of the embankment before blasting. This method is used for fairly narrow fills on peat deposits up to 30 ft deep.

(2) Construction of a narrow embankment on the centre line of the finished work, with subsequent widening on both sides. This method is used for wide embankments or thick deposits of peat in open conditions.

(3) Construction of a narrow embankment on one side of the finished work, with subsequent widening on one side. This method is used for wide embankments or thick deposits of peat where conditions are such that the extrusion of the peat is restricted on one side, e.g. by an existing embankment.

25·13 It is usual in these methods to place one or more rows of charges under the fill to be settled, and in most cases one or two rows of charges outside the edge of the fill to form a space into which the peat under the fill can be moved (Fig. 25·5). It is sometimes necessary to carry out further blasting if the full settlement is not achieved with the first series of charges, and of course with methods (2) and (3) several series of blasts are always necessary.

25·14 Method (1) is illustrated by Fig. 25·6 which shows the settlement of an embankment on part of the Berlin Ring Road at Mehrow[1]. Here up to 16 ft of peat was lying partly under water. Fill was first placed 8 ft wide up to water level, the main charges were then placed and the fill taken up to full height, and subsequently widened to the required width. The object of this procedure was to force out as much peat as possible by the weight of the fill. The subsidiary charges were placed and fired after the main charges had been fired. About 10,000 cu.yd were moved by 2,200 lb. of explosive at a cost of 1s. 6d. per cu.yd which compares favourably with costs of 5s. 0d. and upwards which were quoted in tenders for excavation.

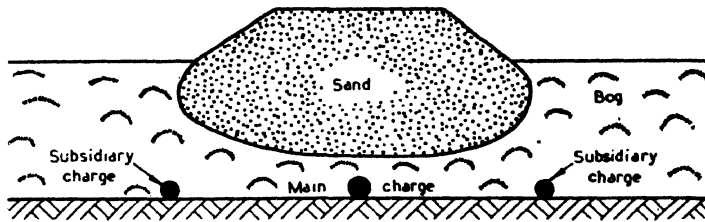
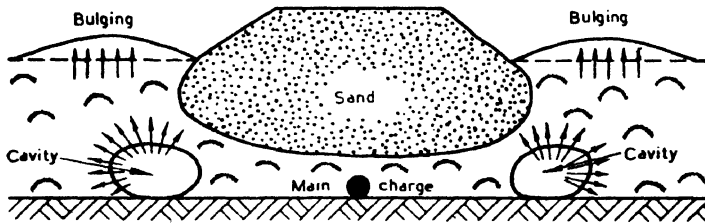
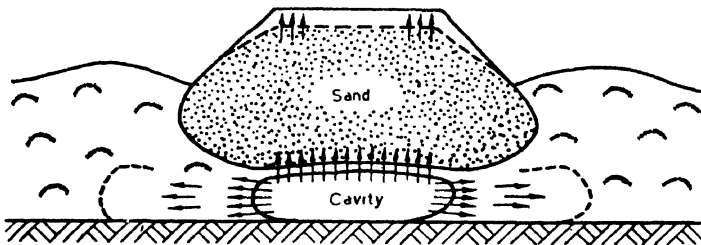
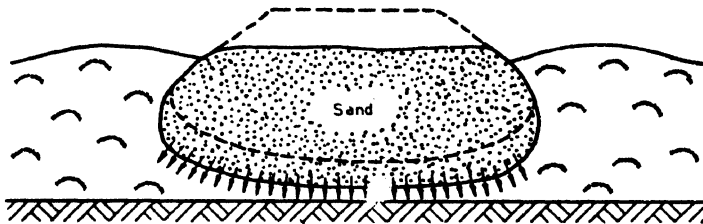
25·15 Method (2) is illustrated by Fig. 25·7 which shows one of the largest operations of this type carried out in Germany[1]. A road near Rangsdorf had to cross 140 yd of bog with depths up to 60 ft. Depths of less than 20 ft were excavated and the remainder settled by five series of blasts. Plate 25·1, A and B, shows the extent of the settlement that may occur from one blast, drops of as much as 20 ft having resulted. In this work 93,000 cu.yd of peat

(a) Embankment after tipping with charges in place

(b) Detonation of subsidiary charges producing cavities

(c) Detonation of main charges producing cavity under embankment lifting momentarily

(d) Final settlement of embankment further displacing the bog

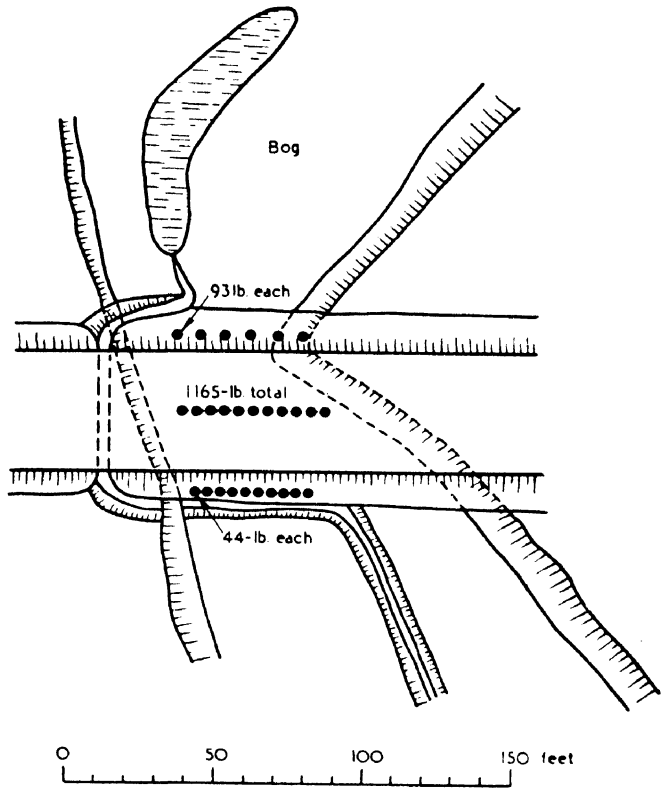
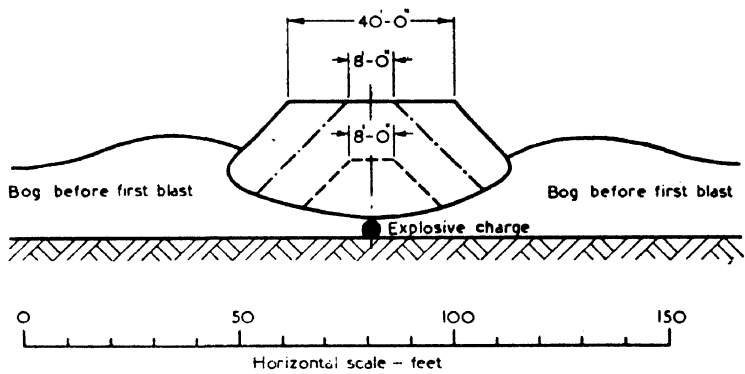
FIG. 25·5 UNDERFILL METHOD OF BOG BLASTING

(a) Plan of the site showing the main charges

(b) Cross-section of embankment

FIG. 25·6 BOG BLASTING AT MEHROW

(a) Before the first series of charges was fired

(b) After the first series of charges was fired showing settlement of nearly 20 ft

THE EMBANKMENT NEAR RANGSDORF

PLATE 25·1

FIG. 25·7 PLAN OF THE MAIN AND SUBSIDIARY CHARGES AT KRUMMEN LANKE (RANGSDORF)

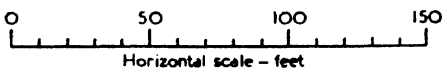
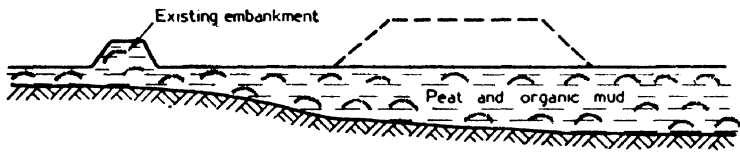
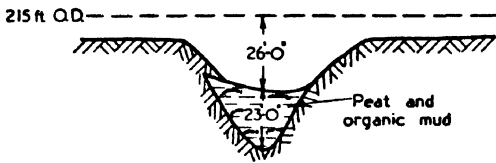
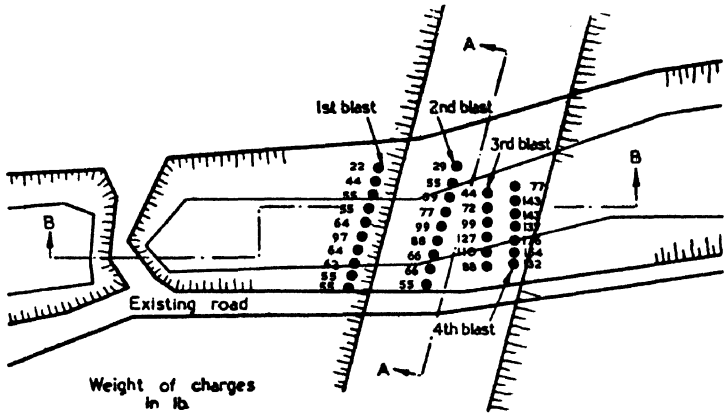



FIG. 25·8 PLAN OF THE SITE AT SEEBURG SHOWING MAIN CHARGES

were displaced, 60 per cent by weight of the fill and 40 per cent by the blasting; 17,000 lb. of explosive were used and the cost was 3½d./cu.yd, while quoted costs for excavation were all over 2s. 0d./cu.yd and in no case covered work below 20 ft deep.

25·16 Method (3) is illustrated by Fig. 25·8 which shows a case at Seeburg in Germany where a road had to cross a narrow boggy depression near an existing road. To avoid pushing the peat against the existing embankment all displacement was in the direction away from it. The settlement was carried out by four series of blasts, the part of the embankment nearest the existing road being dealt with first. The cost was 3s. 6d./cu.yd. All the above costs are for 1937-38.

CONCLUSIONS

25·17 The following suggestions and estimates are put forward tentatively; there will, of course, be exceptional cases where they cannot be applied:—

(1) For small or shallow deposits, excavation will generally be found to be the most economical method of displacement, with trench shooting or displacement by superimposed weight as alternatives.

(2) For deposits between 10 ft and 50 ft deep, displacement by superimposed weight may be used for sites where the peat is fairly soft, possibly supplemented by jetting with toe shooting or torpedo blasting as alternatives.

(3) For deeper deposits or for thickly matted deposits or those overlaid by solid material, some form of blasting is necessary to secure adequate displacement of the peat.

(4) When the peat is lying under water, suction pumps may be used with advantage.

(5) Pre-war costs (1930-1938) show lowest quotations of 2s. 0d. to 5s. 0d./cu.yd. for excavation, 6d./cu.yd as the only quoted cost of softening peat by jetting, 6d. to 9d./cu.yd for deep blasting in the U.S.A. and 3½d. to 3s. 0d./cu.yd in Germany. No details of post-war costs are available.

SUMMARY

25·18 Where an embankment has to be constructed over swampy ground, special methods of construction must be used if undesirable settlements are to be avoided. These include total excavation or partial excavation and partial displacement for shallow peat deposits and jetting or blasting methods for deposits of greater thickness, sometimes exceeding 70 ft. The methods described find their main applications in cases where a road, which is otherwise founded on good ground, has to cross a short length of swampy ground.

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CHAPTER 26

THE STABILITY OF CLAY SLOPES

INTRODUCTION

26·1 The first major contribution to the study of the stability of clay slopes was made by Collin, a French civil engineer who met considerable trouble with slips in constructing canals in about 1846. He concluded that the slip surface was a curve and he realized the great importance of shear tests and of the moisture content of the soil. The importance of Collin's work was not recognized at the time and no further progress was made for over half a century. Apart from the work of Airy (1879), who repeated much that Collin had done, no great contribution to the knowledge of the stability of slopes was made until the 20th century.

26·2 The renewed study of the stability of slopes began almost simultaneously in three countries before the 1914-18 war. The enormous slides which occurred in the construction of the Panama Canal as well as many failures of earth dams in the U.S.A. led to the formation of a special committee of the American Society of Civil Engineers to investigate the problem. In Sweden a series of landslides and subsidences occurred in railway embankments and cuttings, one of which resulted in the loss of 41 lives. This led to the formation of a State Geotechnical Commission to study the problem and, similarly, difficulties met with by German engineers in the construction of the Kiel Canal and other harbour works started researches in Germany into earthwork problems.

26·3 These investigations led to the evolution of various methods of calculating the stability of slopes, one of the most important being the " Swedish Method " which is later discussed in some detail.

26·4 There are two distinct parts to the analysis of the stability of clay slopes. Firstly, undisturbed samples of the soil have to be obtained and tested and the apparent cohesion and angle of shearing resistance found. Secondly, these values are applied to the stability calculations.

26·5 The determination of the strength properties of soil is considered in some detail in Chapter 19. When dealing with soils likely to decrease in strength with time, stiff fissured clays for example, the lowest strength likely to be obtained should be used in the stability analysis. It is not yet possible to assess the probable strength of stiff fissured clays after they have softened but values of shear strength ranging from 2 to 5 lb./sq.in. have been observed.

26·6 The following notes should be considered only as an introduction to the subject. For more detailed information and examples of the application of the stability analysis to practical examples the reader is advised to consult some of the papers listed in the Bibliography.

STABILITY CALCULATIONS—SWEDISH METHOD

26·7 In the analysis of the stability of slopes the assumption is generally made that, when failure occurs, the slip surface is along the arc of a circle. The simplest case of a purely cohesive soil with zero angle of shearing resistance will be considered first.

Case I. Where the soil is purely cohesive with zero angle of shearing resistance.
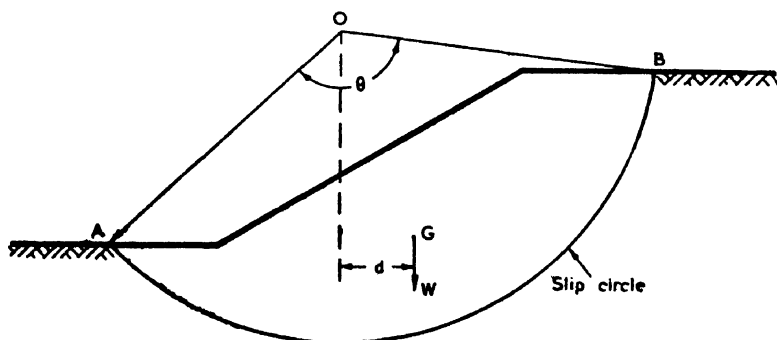
FIG. 26·1 $\phi = 0$ ANALYSIS

26·8 This case is usually referred to as the "$\phi = 0$ analysis."

Consider the stability of the sector of soil cut off by the arc AB of radius r (Fig. 26·1). Assume unit thickness.

Let W be the weight of the sector acting at G the centre of gravity (or centroid) of the sector.

Let s be the shear strength per unit area of the soil.

Let O be the centre of rotation of the circular slip.

Then,

Disturbing moment = Wd

Restoring moment = shear strength x length of arc AB x radius

$= s \times r\theta \times r$

$= sr^2\theta$

When slipping is just about to occur, the disturbing moment is equal to the restoring moment, i.e.

$$Wd = sr^2\theta$$

Alternatively, the factor of safety against slipping (F.O.S.), can be expressed as:

$$\text{F.O.S.} = \frac{\text{Restoring moment}}{\text{Disturbing moment}} = \frac{sr^2\theta}{Wd}$$

An infinite number of arcs may be drawn for any given slope, the one giving the lowest factor of safety being known as the critical circle, the centre of which can be located by Fellenius' construction shown in Fig. 26·2.

26·9 There are two convenient methods of determining the position of the centroid (G) of the sliding sector AB (see Fig. 26·1):—

(1) By dividing the sector into parallel strips and taking moments about any two axes. Since in the stability calculations all that is required is the horizontal distance d (see Fig. 26·1), moments need only be taken about one vertical axis.

(2) By cutting a piece of thin cardboard to the same shape as the sector and suspending it in turn from two points. The intersection of the verticals through the points of suspension gives the position of the centroid.

Case II. Where the soil has both apparent cohesion (c) and an angle of shearing resistance (ϕ)

Slope	i°	β_1°	β_2°
1:0·58	60	29	40
1 : 1	45	28	37
1 : 1·5	33·8	26	35
1 : 2	26·6	25	35
1 : 3	18·4	25	35
1 : 5	11·3	25	37
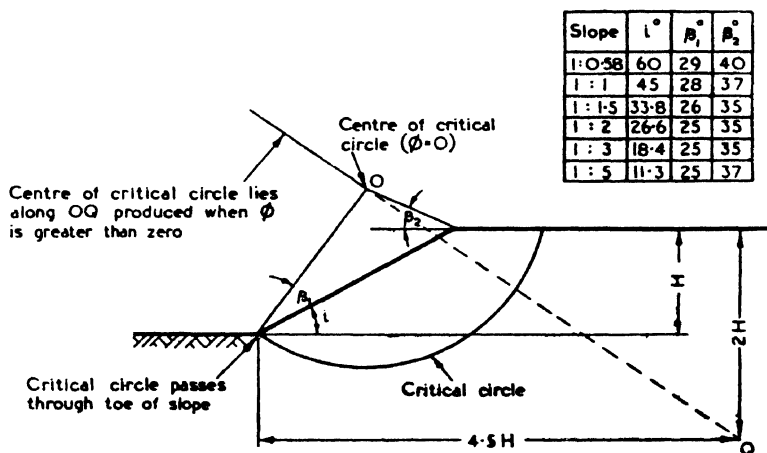
FIG. 26·2 FELLENIUS' CONSTRUCTION FOR THE LOCATION OF CRITICAL CIRCLE

26·10 This case is usually referred to as the c, ϕ analysis. In the case of frictional soils, with an angle of shearing resistance ϕ, the shear strength depends upon the force normal to the plane of rupture. In the case of a circular slip in frictional soils, the normal forces have no moment about the centre of rotation but they affect the value of the shear strength along the surface of rupture and hence alter the restoring moment.

26·11 The method of slices is generally used for calculating the stability in the case of frictional soils. The possible slip circle is drawn as before and the material above the arc AB divided into a convenient number of vertical strips or slices the forces between which are neglected in the analysis. Consider now the stability of any one strip abcd of unit thickness (see Fig. 26·3).
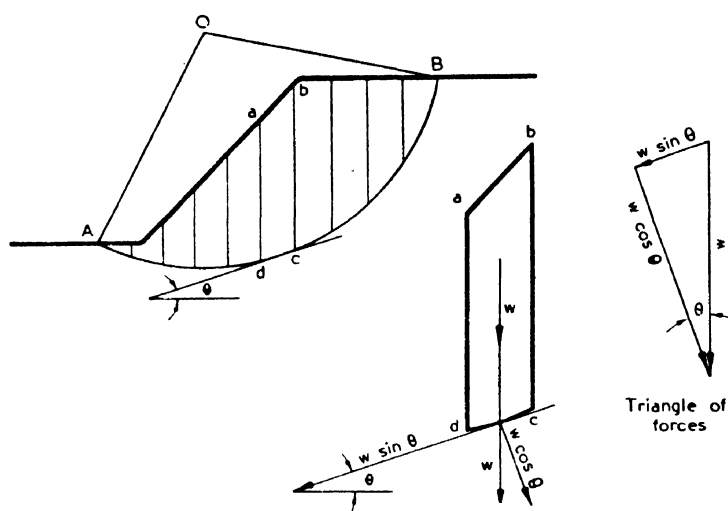
FIG. 26·3 C,ϕ ANALYSIS: METHOD OF SLICES

If w = weight of strip

c = apparent cohesion of soil

ϕ = angle of shearing resistance

l = length of arc of the strip

θ = angle of inclination of the tangent to the horizontal.

Then resolving the weight w in two directions normal and tangential to the arc,

Disturbing force = $w \sin \theta$

Restoring force = $cl + w \cos \theta \tan \phi$

or for the entire sector the factor of safety against slipping can be expressed as

$$\text{F.O.S.} = \frac{\text{Total restoring force}}{\text{Total disturbing force}} = \frac{\Sigma cl + w \cos \theta \tan \phi}{\Sigma\, w \sin \theta}$$

26·12 A tabulated method is generally found to be the most convenient for determining the factor of safety.

26·13 This procedure is repeated until the circle giving the lowest factor of safety (the critical circle) is found. Fellenius' method for aiding the determination of the centre of the critical circle for homogeneous frictional soils is given in Fig. 26·2. The centre of the critical circle lies on the extension of the line through O (the centre when $\phi=0$) from a point Q located at depth 2H from the top of the slope and 4·5H horizontally from the toe, H being the height of the slope.

TABLE 26·1

TABULATED METHOD FOR DETERMINING THE FACTOR OF SAFETY OF THE SLIP CIRCLE

Strip No.	c	ϕ	w	l	θ	w sin θ	cl + w cos θ tan ϕ
1	c_1	ϕ_1	w_1	l_1	θ_1	$w_1 \sin \theta_1$	$c_1 l_1 + w_1 \cos \theta_1 \tan \phi_1$
2	c_2	ϕ_2	w_2	l_2	θ_2	$w_2 \sin \theta_2$	$c_2 l_2 + w_2 \cos \theta_2 \tan \phi_2$
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
n	c_n	ϕ_n	w_n	l_n	θ_n	$w_n \sin \theta_n$	$c_n l_n + w_n \cos \theta_n \tan \phi_n$
						$\Sigma w \sin \theta$	$\Sigma cl + w \cos \theta \tan \phi$

26·14 It must be stressed that Fellenius' rules for locating the centre of the critical circle were based on homogeneous soils and uniform slopes, the top being level. For non-uniform slopes and variable soil conditions the centre can be located only by trial and error methods. Fellenius' method, however, leads to only small errors in many cases and is therefore satisfactory for approximate calculations.

26·15 The method of slices outlined above for the determination of the stability of a slope is useful particularly when the cross-section of the slope does not conform to the simple case and when the soil conditions are very variable.

TENSION CRACKS

26·16 Usually the soil at the top of the slip surface fails in tension resulting in the formation of deep fissures or tension cracks. According to Terzaghi the maximum depth of crack which can occur is given by the expression:—

$$D = \frac{2\,c \tan\left(45^\circ + \frac{\phi}{2}\right)}{\gamma_b}$$

where D = the depth of the tension crack (ft)

c = apparent cohesion of soil (lb./sq.ft)

ϕ = angle of shearing resistance

γ_b = bulk density of soil (lb./cu.ft)

In the case of a purely cohesive soil or one with a very small angle of shearing resistance:—

$$D = \frac{2c}{\gamma_b}$$

26·17 The effect of the tension crack should be taken into account in the stability calculations (Fig. 26·4). This can be done by considering the slip surface producing the restoring moment to end at the tension crack. The entire weight of the sector is used in the calculations as the inclusion of the weight of soil above the tension crack compensates to some extent for any possible hydrostatic pressure due to water filling the tension crack.
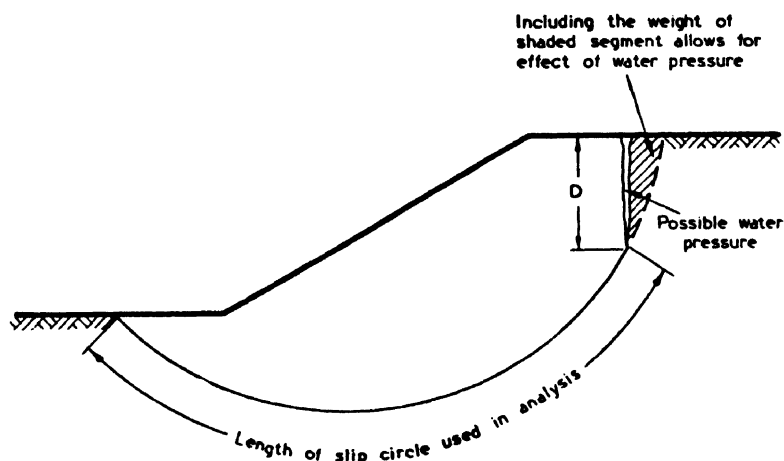
FIG. 26·4 METHOD OF INCLUDING EFFECT OF TENSION CRACK IN STABILITY ANALYSIS

MATHEMATICAL SOLUTION OF THE STABILITY OF UNIFORM HOMOGENEOUS SLOPES

26·18 D. W. Taylor has produced a mathematical solution[1] to the problem of the stability of uniform homogeneous earth slopes that enables the factor of safety of a given slope to be determined from values of the apparent cohesion and angle of shearing resistance of the soil comprising the slope. Alternatively, knowing the soil characteristics of a proposed embankment or cutting, the angle of the slope for a given factor of safety can be obtained. It must be stressed that Taylor's solution is based on the ideal case which rarely occurs in practice, and this should always be borne in mind in the interpretation of results. The method can be very useful, however, in approximate calculations.

26·19 Taylor's curves (Figs. 26·5 and 26·6) give the relationship between a non-dimensional stability number $\left(\frac{c}{FwH}\right)$ and the angle of slope for soils of varying angles of shearing resistance. The factors comprising the stability number $\left(\frac{c}{FwH}\right)$ are:—

c = apparent cohesion of the soil
F = factor of safety with respect to apparent cohesion
w = weight per unit volume of the soil
H = height of the slope.

26·20 The units must be consistent throughout. It will be noticed that Fig. 26·5 gives the stability number for all cases except when $\phi = 0$ with the angle of slope less than 53°. In the latter case the slip circle is theoretically of infinite radius giving for slopes of less than 53° a constant stability number of 0·181.
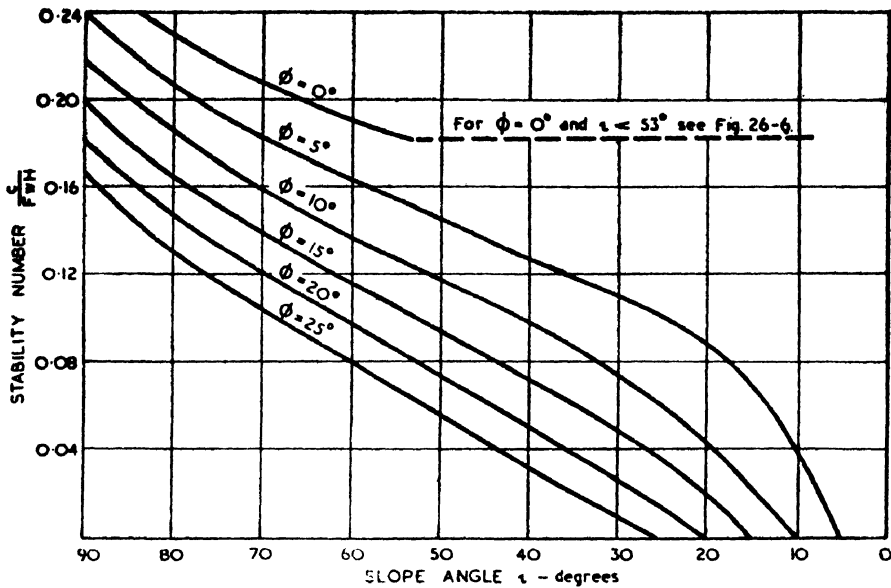
FIG. 26·5 RELATIONSHIP BETWEEN STABILITY NUMBER $\left(\frac{c}{FwH}\right)$ AND SLOPE ANGLE (i) FOR VARIOUS ANGLES OF SHEARING RESISTANCE

In practice the slip circle is restrained at some depth by a layer of stronger soil and the circle will, depending on the conditions, pass through, below or above the toe of the slope. The stability number for these cases can be found from the curves given in Fig. 26·6.

26·21 When the slip circle is limited by some layer of stronger soil or hard stratum at a depth DH (see Fig. 26·6) the full lines in the graph are used. In the case where the circle is constrained to pass through the toe the dotted curves are used. When the stronger layer exists at the level of the base of the slope or above, the slip circle will pass above the toe. A solution can be obtained in this case using the dotted curves as for the second case.

26·22 It will be noticed that the factor of safety given in the stability number refers to the apparent cohesion. To obtain a factor of safety F_T applying to a soil with both an apparent cohesion and an angle of shearing resistance the value of ϕ is replaced by ϕ_D where

$$\phi_D = \frac{\phi}{F_T}$$

Examples showing the use of Taylor's curves are given in the appendix to this chapter.
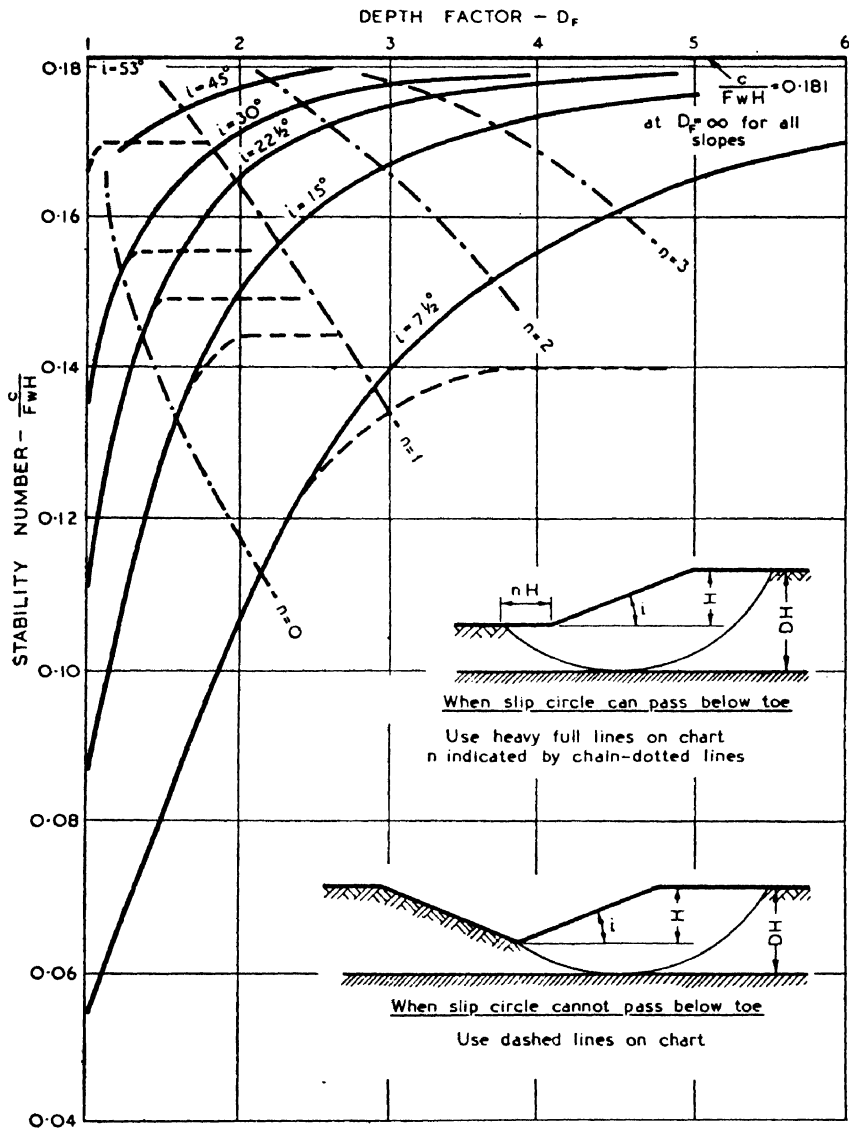
FIG. 26·6 RELATIONSHIP BETWEEN DEPTH FACTOR (D_F) AND STABILITY NUMBER $\left(\frac{c}{FwH}\right)$ FOR $\phi = 0$ AND $i < 53°$

REMEDIAL MEASURES

26·23 There are a number of ways in which the factor of safety of a clay slope against slipping can be increased or an existing slip stabilized.

26·24 Slopes composed of highly fissured clays are often rendered unstable by the action of surface or seepage water penetrating down the fissures and softening the clay. Tension cracks in particular can act as traps for surface runoff leading the water down into the possible slipping zone. Drainage measures should be taken, therefore, to intercept surface and seepage water and lead it away from the slope. An intercepting drain is often installed along the top of the slope and deep counterfort drains up the face. Shallow herring-bone drains are provided to trap surface water running down the slope and lead it to the deep counterfort drains. The latter generally consist of deep trenches cut into the face of the slope at intervals of about 15 to 50 ft to the depth of the possible critical circle, piped and filled with rubble or suitable permeable material. The herringbone drains, or laterals as they are sometimes called, are usually of much shallower depth (about 2 ft) and in many cases have no drain pipe although the installation of the latter in the trench bottom is considered preferable.

26·25 Apart from their drainage action the deep counterfort drains, as their name implies, act as buttresses increasing the stability of the slope.

26·26 The factor of safety of unstable slopes can be increased by the construction of berms of soil or rock along the toe of the slope. The removal of soil from the top of the slope, in the form of a benching, or lessening the gradient of the slope will also improve stability.

26·27 Sheet piling driven into the ground near the toe of the slope will force the possible critical circle to go down deeper and will in general increase the factor of safety against slipping. This method of improving the stability of the slope may prove in practice to be more expensive than the other methods and economic factors will always have to be considered before any particular remedial measure is adopted.

26·28 Vegetation of suitable types has been successfully employed in stabilizing slopes[2] [3]. Apart from reducing the moisture content of the soil (and hence increasing the shear strength), the network of roots acts as a reinforcement in the soil. Fast-growing trees such as the poplar are of particular value in reducing the moisture content of the soil but care should be exercised in locating the trees as the removal of moisture by the roots from the subgrade of roads is likely to cause settlement. For this reason, fast-growing trees should not be planted within about 50 ft of the carriageway. Vegetation can also act as a blanket on the surface layers of soil by limiting the softening effects of frost and reducing erosion.

APPENDIX TO CHAPTER 26

Examples showing the use of Taylor's Curves

26·29 A cutting 40 ft deep is made in clay having the following characteristics:— $c = 750$ lb./sq.ft, $\phi = 10°$, $w = 110$ lb./cu.ft. The angle of the slope is 40°. What is the factor of safety with respect to the cohesion? What new value of cohesion is necessary to give a true factor of safety of the same magnitude?

From Fig. 26·5, for $\phi = 10°$ and $i = 40°$, $\frac{c}{FwH} = 0{\cdot}098$

$$\frac{c}{FwH} = 0{\cdot}098 = \frac{750}{F \times 110 \times 40}$$

$$\frac{750}{0{\cdot}098 \times 110 \times 40} = 1{\cdot}74$$

$$\phi_D = \frac{\phi}{F_T} = \frac{10}{1{\cdot}74} = 5{\cdot}75°$$

For $\phi = 5{\cdot}75°$, $i = 40°$, Fig. 26·5 gives $\frac{c}{FwH} = 0{\cdot}123$

$$\therefore\ 0{\cdot}123 = \frac{c}{1{\cdot}74 \times 110 \times 40}$$

$$c = 0{\cdot}123 \times 1{\cdot}74 \times 110 \times 40 = 942 \text{ lb./sq.ft.}$$

26·30 A cutting 25 ft deep is to be made through a clay of which the value of cohesion corrected to allow for tension cracks is 400 lb./sq. ft, w = 120 /lb. cu.ft, $\phi = 7{\cdot}5°$. A factor of safety of 1·5 is specified.

What is the allowable slope?

$$\phi_D = \frac{\phi}{F_T} = \frac{7{\cdot}5}{1{\cdot}5} = 5°$$

$$\frac{c}{FwH} = \frac{400}{1{\cdot}5 \times 120 \times 25} = 0{\cdot}089$$

From Fig. 26·5, when $\frac{c}{FwH} = 0{\cdot}089$ and $\phi_D = 5°$

$$i = 20{\cdot}3°$$

That is, a slope of 1:2·7 will be required.

26·31 A cutting 45 ft deep is to be made in a highly cohesive soil which is 60 ft thick and is underlaid by a ledge of sandstone. The shear strength of the clay is 700 lb./sq.ft and can be assumed constant down to the sandstone. The unit weight of the soil is 100 lb./cu. ft. What factor of safety exists for 1:1 slopes? What slope angle is required for a factor of safety of 1·2?

Since $\phi = 0$ and $i < 53°$, Fig. 26·6 is used.

Depth factor $D_F = \frac{60}{45} = 1{\cdot}33$

Slope angle $i = 45°$

From Fig. 26·6, with D_F = 1·33 and i = 45°. $\frac{c}{FwH}$ = 0·171

$$\frac{c}{FwH} = 0{\cdot}171 = \frac{700}{F \times 100 \times 45}$$

$$\text{or } F = \frac{700}{0{\cdot}171 \times 100 \times 45} = 0{\cdot}91$$

From Fig. 26·6, n = 0·5 (approx.). Hence nH = 0·5 x 45 = 22·5 ft. Thus for a slope of 1:1 the cutting will be unstable and the slip circle will cut the surface about 22·5 ft from the toe.

For a factor of safety of 1·2

$$\frac{c}{FwH} = \frac{700}{1{\cdot}2 \times 100 \times 45} = 0{\cdot}130$$

From Fig. 26·6, for D_F = 1·33 and $\frac{c}{FwH}$ = 0·130

$$i = 19°$$

That is, a slope of 1:2·9 will be required.

SUMMARY

26·32 The stability of a clay slope may be analysed by means of the " Swedish method " evolved as a result of the studies of the Swedish Geotechnical Commission, which requires a knowledge of the apparent cohesion and the angle of shearing resistance of the soil. The Swedish method of analysis involves the selection of various possible slip circles. The factor of safety of each slip circle is determined, the one giving the lowest factor of safety being known as the critical circle. Fellenius' rules for locating the centre of the critical circle are given.

26·33 Taylor's mathematical solution of the stability of earth slopes is explained briefly and three examples illustrating the use of the method are given in an appendix.

26·34 Possible remedial measures that can be adopted to improve the stability of slopes include drainage of the slope, construction of a berm at the toe of the slope and the cutting of a benching at the top, and the driving of sheet piling.

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BALANCE WEIGHING TO 7 KGM

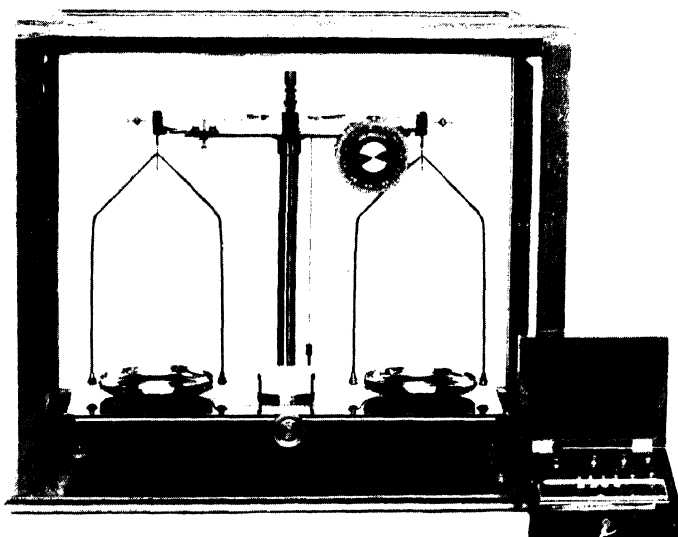
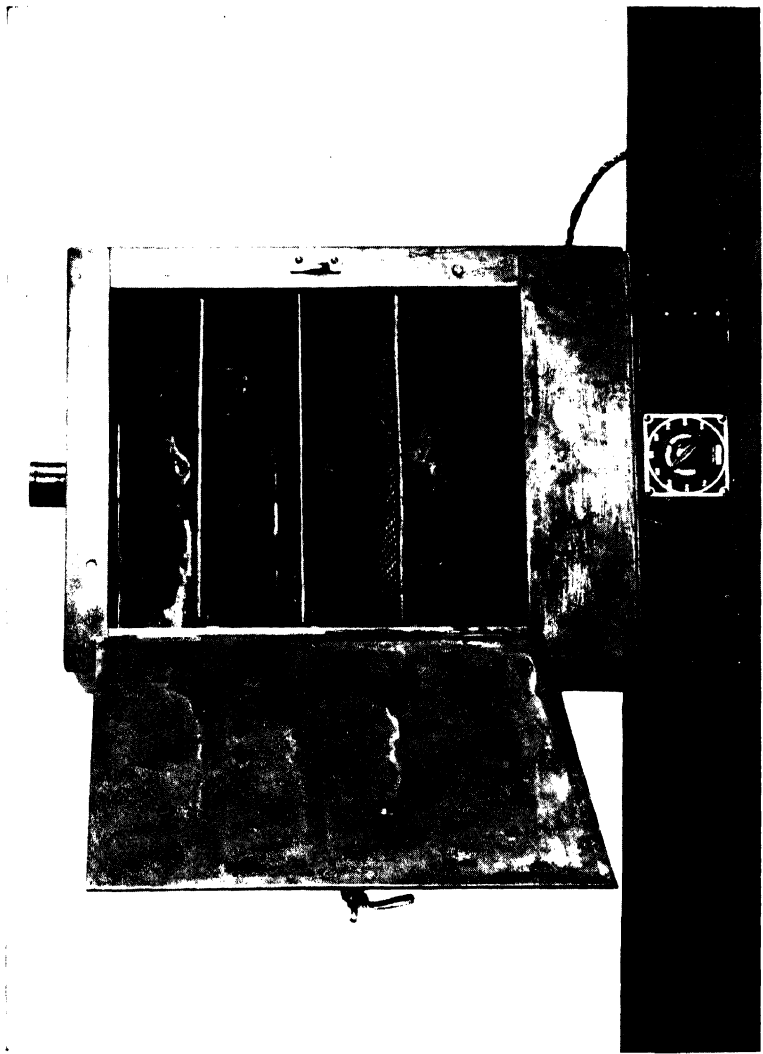
PLATE 27·1

CHAINDIAL BALANCE WEIGHING TO 150 GM

PLATE 27·2

THERMOSTATICALLY CONTROLLED DRYING OVEN

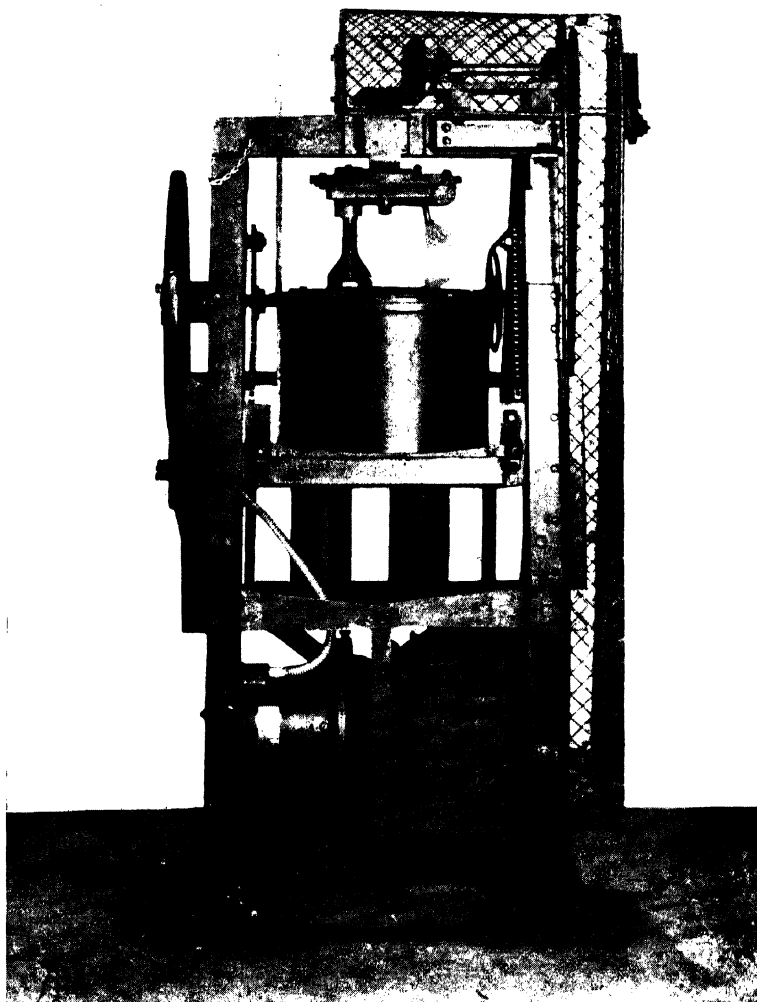



PLATE 27·3

MECHANICAL MIXER
for stony soils (capacity 7 kgm)

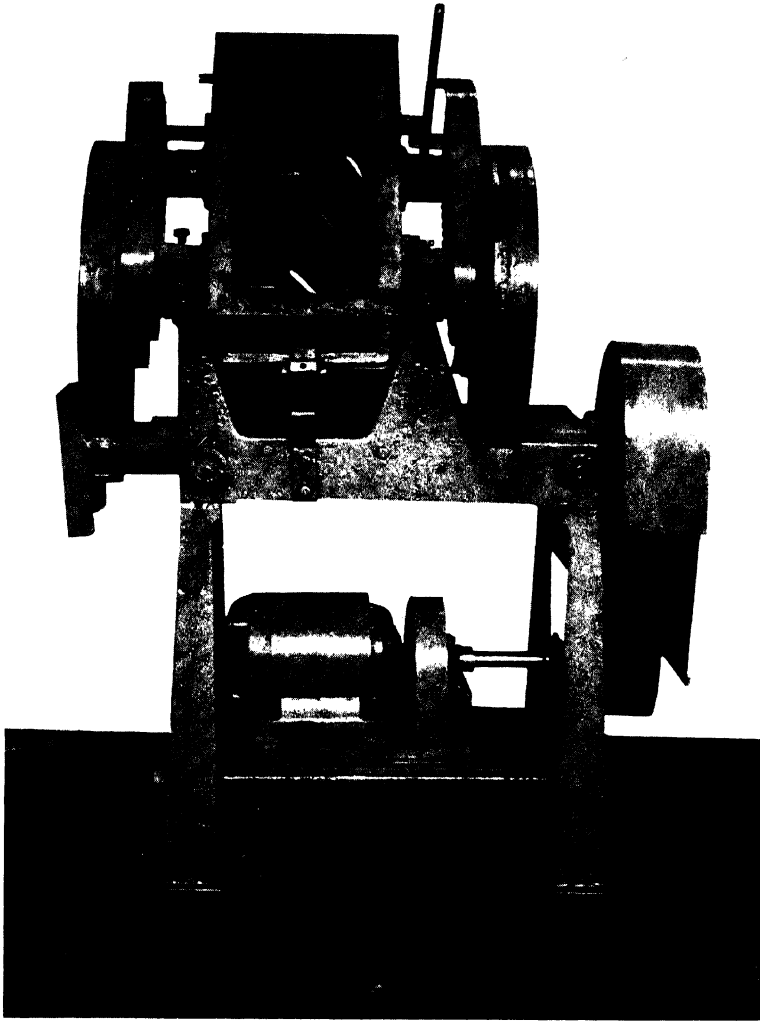
PLATE 27·4

MECHANICAL MIXER
for fine-grained soils (capacity 6 kgm)

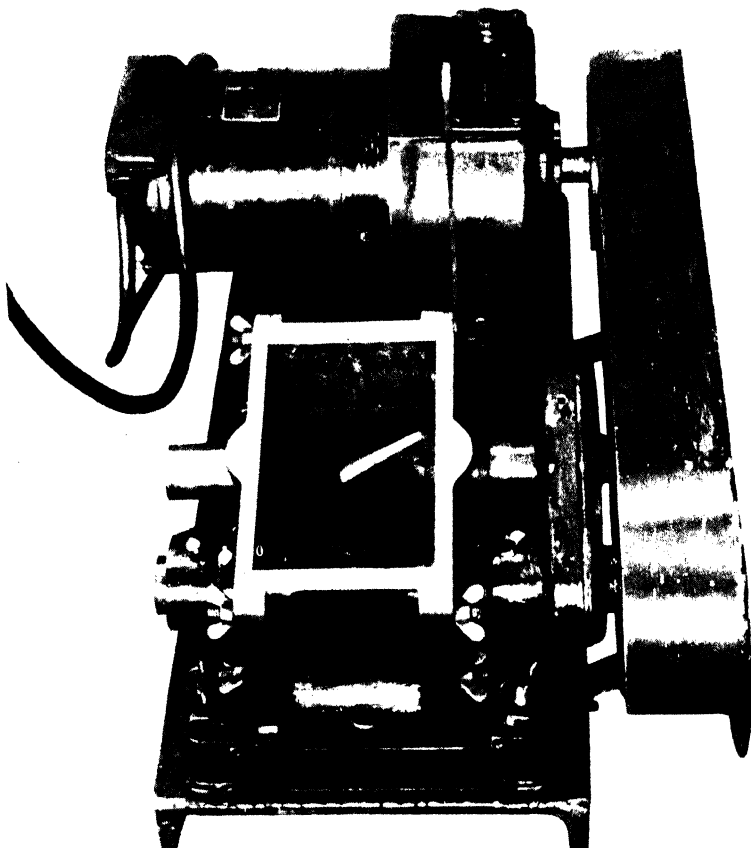
PLATE 27·5

MECHANICAL MIXER
for fine-grained soils (capacity 700 gm)

PLATE 27·6

CHAPTER 27

EQUIPMENT FOR SOIL-TESTING LABORATORY

INTRODUCTION

27·1 In selecting the equipment to be purchased for a soil-testing laboratory consideration has first to be given to the scope of work for which the laboratory is intended, and a decision reached as to the tests to be carried out. This list of equipment has therefore been divided into two parts: the first part (1) gives general equipment required for a soil-testing laboratory and the second part (2) gives the equipment required for each test.

27·2 In part (2) there is, of course, some repetition of the apparatus listed in part (1), but unless more than one type of the article is suggested in the list of general equipment no dimensions or measurements are given, when it is mentioned in part (2). The balances given in part (2) all refer to those listed in part (1) except for the 25-kgm balance required only for the California bearing ratio test. Special equipment required for only one type of test is not mentioned in part (1).

27·3 This list is intended merely to act as a guide in the purchasing of equipment and the numbers or quantities of each item are left to the discretion of the engineer, who alone knows the size and scope of the laboratory he is planning.

EQUIPMENT REQUIRED

Test	Equipment required
(1) BASIC EQUIPMENT (i) **General equipment**	Pestle (rubber-covered) and mortar, 11-cm. diam. „ „ „ „ 17-cm. diam. 10-in. diam. desiccators 4-in. steel spatulas with blade ¾ in. wide. 6-in. „ „ „ „ „ „ 8-in. „ „ „ „ „ „ 0°-110°C. thermometers 0°-250°C. „ 0°-50°C. „ 500-ml. beakers 9-cm. diameter funnels Double funnel holders 1,000-ml. wash bottles 1,000-ml. graduated cylinders 500-ml. „ „ 250-ml. „ „ 100-ml. „ „

Test	Equipment required
(i) **General equipment** (contd)	25-ml. graduated cylinders 10-ml. „ „ 12·5-cm diam. Büchner funnels and 1,000-ml. receivers 50-ml. density bottles Glass weighing bottles 55 mm. high, 25 mm. in diam., with ground glass stopper Glass weighing bottles 75 mm. high, 35 mm. in diam., with ground glass stopper 500-ml. conical flasks 350-ml. „ „ 50-ml. burettes Burette stands Tripods 8 in. high, 5-in. top 5-in. square gauzes 14-cm. diameter evaporating dishes 3-in. diameter watch glasses Crucible tongs Rubber tubing $\frac{1}{2}$-in. overall diam., $\frac{1}{4}$-in. internal diam. Glass tubing $\frac{5}{16}$-in. overall diam. Solid glass rod, $\frac{5}{16}$-in. diam. Blue litmus paper Bats-wing burner Bunsen burners Filter paper 12·5-cm. diam. Paraffin wax
(ii) **Balances**	A. Balance weighing to 7 kgm, accurate to 1 gm, and set of weights (Plate 27·1) B. Semi-automatic balance weighing to 500 gm, accurate to 0·1 gm, and set of weights C. Chaindial balance weighing to 150 gm, accurate to 0·01 gm, and set of weights (Plate 27·2) D. Chemical balance weighing to 100 gm, accurate to 0·001 gm, and set of weights
(iii) **Sieves**	B.S. sieves of the following sizes:— $1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., $\frac{3}{16}$-in., Nos. 7, 14, 25, 36, 52, 72, 100 and 200, with base and cover
(iv) **Ovens**	Thermostatically controlled drying ovens capable of maintaining a temperature of 105°-110°C. (Plate 27·3)

Test	Equipment required
(iv) **Ovens** (contd)	Oven for field work. Internal dimensions 12 in. by 12 in., capable of maintaining temperature of 105°-110°C. Calor gas or oil-burning type
(v) **Sample tins**	Watertight sample tins with lever lids 3½-in. diam., 2¾ in. high (1 lb. size) Aluminium ointment tins (2-oz size).
(vi) **Jacks**	Car-type hydraulic jack, closed height 6-7 in., run-out 4½-5 in. with flat head
(vii) **Mixers** (Plates 27·4, 27·5 and 27·6)	Mechanical mixer, capacity 14 lb. (7 kgm) (for stony soils) " " " 6 kgm (for fine-grained soils) " " " 700 gm
(viii) **Trays**	Stainless steel trays, 12-in. diameter, 2 in. high Aluminium trays 2 ft x 2 ft x 2½ in. deep
(2) EQUIPMENT FOR SEPARATE TESTS I. **Preparation of samples for testing**	Balance of type B B.S. sieves Nos. 7 and 36 Pestle (rubber-covered) and mortar Riffle box
II. **Determination of moisture content** (a) *Oven-drying method*	(I) *For fine-grained soils* Aluminium ointment tins or glass weighing bottles (75 mm. x 35 mm.) if preferred Balance of type C Thermostatically controlled oven Desiccator Spatula (II) *For coarse-grained soils* 1-lb. size sample tins Balance of type B Thermostatically controlled oven Desiccator Spatula (For field work—Calor gas *or* oil-burning oven)

Test	Equipment required
(b) *Sand-bath method*	(I) *For fine-grained soils* Aluminium ointment tins Balance of type B Sand bath approx. 7 in. in diam. with sand $\frac{1}{2}$ in. deep Apparatus for heating sand bath, e.g. kerosene pressure stove Desiccator Spatula (II) *For coarse-grained soils* 1-lb. size sample tins Balance of type B Sand bath and apparatus for heating it (as above) Desiccator Spatula
(c) *Pycnometer method*	A pycnometer (made from 2-lb. glass fruit jar with brass conical top to fit jar and rubber gasket) Balance of type A Glass rod 12 in. long, $\frac{1}{4}$-in. diameter Thermometer 0°-50°C. (32°-122°F.) Fountain-pen filler (optional)
III. **Liquid limit test**	Liquid limit device and tools Piece of polished plate glass about 2 ft square and $\frac{3}{8}$ in. thick *or* evaporating dish 14 cm. in diam. Spatulas Apparatus for determination of moisture content
IV. **Plastic limit test**	Piece of polished plate glass 2 ft square, $\frac{3}{4}$ in. thick Spatulas Apparatus for determination of moisture content
V. **Determination of specific gravity** (a) *Laboratory method*	Density bottle, 50-ml. capacity Water bath maintained at 20°C. Vacuum desiccator Thermostatically controlled drying oven Balance of type D

Test	Equipment required
(b) *Field method*	Pycnometer (see moisture content, method (c)) Thermostatically controlled drying oven Balance of type A Desiccator Glass rod, 12 in. long, $\frac{1}{4}$-in. diam. Thermometer 0°-50°C.
VI. **Determination of particle-size distribution** (a) *Pipette method*	Sampling pipette fitted with pressure and suction inlet, 10ml. capacity Glass boiling tubes 5 cm. in diam., 34 cm. long, 500-ml. capacity Glass weighing bottles (55 mm. by 25 mm.) Constant-temperature bath maintaining temperature of 25°C. in which boiling tube can be immersed up to 500-ml. mark High-speed (not < 5000 r.p.m.) mechanical stirrer with cup to fit and wire baffle B.S. sieves $1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., $\frac{3}{16}$-in., Nos. 7, 25, 72 and 200, and receiver Balances of type A and D 50-ml. pipette Thermostatically controlled drying oven Stop-clock or watch Desiccator Porcelain evaporating dishes Filter funnel Büchner funnel and receiver Wash bottle Filter paper A.R. sodium oxalate A.R. hydrochloric acid 20-vol. solution of hydrogen peroxide Blue litmus paper
(b) *Hydrometer method*	Hydrometers—one long-stemmed and one short-stemmed 1000-ml. graduated cylinder Thermometer 0°-50°C. High-speed mechanical stirrer (as above) Pestle (rubber-covered) and mortar B.S. sieves:—$1\frac{1}{2}$-in., $\frac{3}{4}$-in., $\frac{3}{8}$-in., and $\frac{3}{16}$-in. Nos. 7, 25, 72 and 200, and receiver Balance of type C

Test	Equipment required
(b) *Hydrometer method* (contd)	Thermostatically controlled drying oven Stop-clock or watch Desiccator Centimetre scale Porcelain evaporating dish Filter funnel 50-ml. pipette Büchner funnel and receiver Wash bottle Filter paper A.R. sodium oxalate A.R. hydrochloric acid 20-vol. solution of hydrogen peroxide Blue litmus paper
VII. **Determination of carbonate content**	Collins' calcimeter Balance of type D Watch glasses Spatulas Hydrochloric acid
VIII. **Determination of organic content**	Burettes and stands 500-ml. conical flasks 250-ml. graduated cylinder 25-ml. ,, ,, 10-ml. ,, ,, Watch glass Wash bottle White tiles A.R. ferrous sulphate A.R. potassium dichromate 85% phosphoric acid Conc. sulphuric acid Diphenylamine
IX. **Compaction test**	
(a) *B.S. compaction test*	Cylindrical metal mould, internal diam. 4in., 4·59 in. high, volume $\frac{1}{30}$ cu. ft, with detachable base plate and removable extension 2·5 in. high. Metal rammer with 2-in. diam. circular face, weighing 5·5 lb., with arrangement to control drop to 12 in. Balance of type A Spatula 12-in. steel straight-edge $\frac{3}{4}$-in. B.S. sieve

Test	Equipment required
(a) *B.S. compaction test* (contd)	Apparatus for extracting specimen from mould, including hydraulic jack Mechanical mixer Apparatus for determination of moisture content
(b) *Modified* (*A.A.S.H.O.*) *compaction test*	Cylindrical metal mould as above Metal rammer with 2-in. diam. circular face, weighing 10 lb., with arrangement to control drop to 18 in. Balance of type A Spatula 12-in. straight-edge $\frac{3}{16}$-in. B.S. sieve Apparatus for extracting specimen from mould, including hydraulic jack Mechanical mixer Apparatus for determination of moisture content
X. **Determination of dry density**	
(a) *Sand-replacement method*	Sand-pouring cylinder with conical funnel and tap Metal tray 1 ft square with 4-in. diam. hole in centre (optional) Dry, clean, closely graded natural sand passing No. 25, retained on No. 52 B.S. sieve Suitable tool for excavating hole in soil, e.g. spatula, iron spoon or trowel Cylindrical metal calibrating container, internal diam. 4 in., internal depth 5 in., with lip 1 in. wide and $\frac{1}{8}$ in. thick surrounding open end Balance of type A Apparatus for determination of moisture content
(b) *Core-cutter method*	Cylindrical steel core-cutter 5 in. long, 4-in. internal diam., wall thickness $\frac{1}{8}$ in., bevelled at one end Steel dolly 1 in. high, 4-in. internal diam., wall thickness $\frac{1}{4}$ in., fitted with lip Steel rammer with wood or steel handle

Test	Equipment required
(b) *Core-cutter method* (contd)	Apparatus for extracting sample from rammer, with reversible top plate Hydraulic jack Balance of type A Spatula Steel foot rule, graduated in 0·01 in. Grafting tool or spade, and pickaxe Apparatus for determination of moisture content
(c) *Water-displacement method*	Cylindrical metal container, fitted with a syphon tube Balance of type A Graduated cylinders of 200-ml., 500-ml. and 1000-ml. capacity Paraffin wax Equipment for melting wax Apparatus for determination of moisture content
XI. **Unconfined compression test**	Portable apparatus with set of 4 springs and masks Specimen extractor and coning tool Auger, rod and 2 extensions, fitted with adaptor for cutters 1½-in. internal diam. steel cutter, with relieved internal diam. Cutter as above but unrelieved Spatula
XII. **California bearing ratio test** (a) *Laboratory test*	Loading frame, jack preferably screw-type, load-measuring device, standard plunger and dial gauge to measure penetration. Set of moulds with base plates, collars, cutting edges, displacer discs, perforated discs and stems (Set of thin-walled steel moulds also desirable for undisturbed specimens) Tripods and dial gauges 'C' spanner Set of 5-lb. surcharge weights B.S. sieves, ¾-in. and $\frac{3}{16}$-in. Mechanical mixer Balance to weigh to 25 kgm 5½-lb. rammer

Test	Equipment required
(a) *Laboratory test* (contd)	10-lb. rammer Galvanized iron soaking tank Straight-edge
(b) *In situ test*	Jack, load-measuring device and standard plunger to act from underside of a loaded vehicle Datum frame and dial gauge to measure penetration of plunger. Set of 5-lb. surcharge weights and one 10-in. diam. surcharge plate
XIII. **Plate-bearing test** (a) *Light equipment*	Jack and load-measuring device to act from underside of a loaded vehicle providing 3-5-tons reaction (can be combined with equipment for *in situ* C.B.R. test) 12-in. diam. steel plate 18-in. „ „ „ Datum frame and 2-4 dial gauges Plaster of Paris
(b) *Heavy equipment*	Loaded equipment as for light equipment but for maximum of 25-50-tons reaction Sets of steel plates, 12-30-in. diam. Datum frame and four dial gauges Plaster of Paris
XIV. **Shear box test**	Machine consisting of a shear box, built-in screw jack, proving ring, stirrup and weights up to 150 lb., dial gauge, two perforated and two plain grids, two porous stone or metal plates Motor drive and interchangeable gear trains for above
XV. **Triaxial compression test**	Triaxial pressure cell Loading machine (preferably motor-driven) Load-measuring device Pressure gauge Manometer
XVI. **Consolidation test**	Moulds with dial gauges for testing 3-in. diam. samples Loading frame

Test	Equipment required
XVII. **Unconfined compressive strength test for stabilized soil**	Balance of type A Balance of 25 kgm capacity reading to 5 gm B.S. sieves, 1½-in. ¾-in. and No. 7. Compression testing machines of 5 and 50 tons capacity. Mechanical mixers Constant volume split moulds, fitted with flanged plungers, suitable for preparing cylindrical specimens 2-in. diam. and 4 in. high, 4-in. diam. and 8 in. high, and 6-in. diam. and 12 in. high. Ejecting plungers for use with above moulds Metal rammer, as used in B.S. compaction test Metal rammer, as used in modified A.A.S.H.O. compaction test 1,000-ml. measuring cylinder 100-ml. „ „ Steel tamping rod Straight-edge Spatula Paraffin wax Apparatus for determination of moisture content
XVIII. **Capillary water absorption test**	Apparatus for the unconfined compressive strength test for stabilized soils Balance of type B Absorption tank to give 2-mm. head of water
XIX. **Soil survey**	*For setting out* Theodolite (optional) Level (quickset type) Levelling staff Level book 100-ft linen tape or chain, and 50-ft linen tape Surveying arrows with markers 7-lb. sledge hammer Ranging rods Axe Sickle Supply of pegs painted white at top

Test	Equipment required
XIX. **Soil survey** (contd)	*For boring* Post-hole augers (4-in. or 5-in.) and extensions 2-in. twist auger Crowbar (capable of being connected with auger extension) Stillson wrenches or tommy bars (for fitting extensions to augers) Spatulas Spades (for clayey ground) Shovels (for sandy ground) Picks Sample tins Boring record sheets *General* 10- to 12-cwt van Boxes about 15-in. cube to carry sample tins Sacks Pegs String and rope Rags Tin of vaseline Bottle of hydrochloric acid Packing material Pencils and notebooks Chinagraph pencils Squared paper Set squares Scales Camera

CHAPTER 28

DEFINITIONS AND SYMBOLS

INTRODUCTION

28·1 The soil mechanics and civil engineering terms used in this book have, in general, been defined in the text. For reference purposes, however, a list of the more important of these terms, together with their definitions, is given below. Terms of a purely geological nature have been omitted, but many of these, of interest to the road engineer, are defined in Chapter 6.

28·2 A list of symbols, and the quantities which they have been used to represent, is also included. Fig. 28·1 (p. 529) shows the methods of indicating rock and soil types in diagrams.

DEFINITIONS

28·3

Acid soil. A soil having a pH value less than 7·0.

Alkaline soil. A soil having a pH value greater than 7·0.

Ballast. Stone or gravel mixtures of irregular unscreened sizes which may also contain smaller material and sand.

Base. That part of the construction resting upon, and through which the load is transmitted to the sub-base, subgrade or supporting soil.

Bitumen emulsion. A colloidal suspension of bitumen in water.

Black cotton soil. A brown or black clay soil in which volume changes due to swelling or shrinkage are particularly marked.

Boulder clay. A deposit of unstratified clay or sandy clay of glacial origin containing subangular stones of various sizes scattered irregularly throughout its mass. The stones are not necessarily all of " boulder " size.

Boulders. Stones larger than 200 mm. in size.

Brickearth. A soil containing clay, silt and sand usually of a buff or biscuit colour, homogeneous, without any structure, and suitable for brickmaking. Found mainly in the Thames estuary and S.E. England.

Bulk density. The weight of a material (including solid particles and any contained water) per unit volume including voids.

Cement-stabilized soil, bitumen-or tar-stabilized soil, chemically stabilized soil. Soil in which stabilization has been assisted by the addition of cement, bitumen, tar or chemicals.

Chalk. A soft limestone consisting principally of the remains of marine animals.

Clay. Colloidally fine, complex silicates formed by the natural decomposition of igneous rocks.

Clay fraction. That fraction of a soil composed of particles smaller in size than 0·002 mm.

Cobbles. Stones between 60 and 200 mm. in size.

Coefficient of uniformity. A term indicating the grading of a material; it is the ratio of the sieve size through which 60 per cent of the material passes to the sieve size through which 10 per cent passes.

Cohesive soils. Soils consisting of the finer products of rock weathering. The cohesion is derived from the large number of water films associated with the fine-grained particles in the soil.

Compaction. The process whereby the soil particles are constrained, by rolling or other means, to pack more closely together, thus increasing the dry density of the soil.

Consolidation. The process whereby soil particles are packed more closely together by the application of continued pressure over a period of time, e.g., an embankment under its own weight or the soil under a building.

Course. Prepared material placed to form a continuous layer of the pavement.

Cut-back bitumen. Bitumen which has been rendered fluid at atmospheric temperature by the addition of a suitable diluent such as white spirit, kerosene or creosote.

Drainage. Natural or artificial means for the removal of water from the surface or subsoil of an area, usually by means of gravitation.

Dry density. The weight of the dry material after drying to constant weight at 105°C. (221°F.), contained in unit volume of moist material.

Dry density/moisture content relationship. The relationship between dry density and moisture content of a soil when a given amount of compaction is applied.

Equilibrium moisture content. The moisture content at any point in a soil after moisture movements have ceased.

Fill. Excavated soil, rock or refuse when dumped for the purpose of filling a depression or raising a site above the natural surface level of the ground.

Formation. The surface of the ground in its final shape after completion of the earthworks, and of consolidation, compaction or stabilization *in situ*.

Grading. See particle-size distribution.

Gravel. Rounded or water-worn stones of irregular shape and size occurring in natural deposits with or without some finer material.

Gravel fraction. That fraction of a soil composed of particles between 2·0 and 60 mm. in size.

Gravitational water. The water which moves downwards under the action of gravity, from the soil surface to the water-table.

Ground water. The water contained in soil below the water-table.

Hardpan. A horizon of accumulation that has been thoroughly cemented to an indurated, rock-like layer that will not soften when wet.

Held water. The water retained in the soil structure above the water-table by surface tension and adsorption forces.

Hoggin. A deposit of sand and gravel as occurring in nature and containing a proportion of clay which is sufficient to hold the mass together.

Lateritic soil. Tropical soil in which the weathering processes have resulted in an accumulation of sesquioxides, particularly iron.

Leaching. The process by which soluble material in soil is removed by the percolation of water.

Liquid limit. The moisture content at which the soil passes from the plastic to the liquid state as determined by the liquid limit test.

Liquidity index. The difference between the natural moisture content and the plastic limit expressed as a percentage ratio of the plasticity index.

Marl. Soil consisting of a natural mixture of calcareous clay or calcareous silty clay.

Maximum dry density. The dry density of a soil obtained by a specified amount of compaction at the optimum moisture content.

Mechanically stabilized soil. Soil to which imported soil or aggregate has been added to obtain a desired particle-size distribution and which has been compacted to a desired density.

Modulus of deformation. The approximate modulus of elasticity determined from the slope of the straight line extending from the origin to a given strain on a stress/strain curve. Usually expressed in lb./sq.in.

Modulus of subgrade reaction. The slope of the straight line extending from the origin to a given deformation on a stress/deformation curve obtained from field bearing tests. Usually expressed in lb./sq.in./in.

Moisture content. The loss in weight, expressed as a percentage of the dry material, when a soil is dried to constant weight at 105°C. (221°F.).

Non-cohesive soil. Soil consisting of the coarser products of rock weathering in which the cohesive bonds mainly associated with the smaller fractions are largely absent.

Optimum moisture content. That moisture content at which a specified amount of compaction will produce the maximum dry density.

Particle-size distribution (Grading). The percentages of the various grain sizes present in a soil as determined by sieving, sedimentation, elutriation or other means.

Pavement. The whole of the artificial construction made to support traffic above the subgrade.

Pavement, flexible. A pavement of inconsiderable flexural rigidity or tensile strength.

Pavement, rigid. A pavement developing considerable local flexural rigidity by reason of the " tensile " strength of one or more of its courses.

Peat. Dark, fibrous, spongy soil of vegetable origin.

pF. The pF value of the held water in soil is equivalent to the common logarithm of the suction expressed in centimetres of water.

pH. The pH of a soil is the negative logarithm of the hydrogen-ion concentration in an aqueous suspension of the soil.

Pitching. Large stones, usually from 7 to 12 in. in depth, when placed by hand and compacted by rolling to form a stable base coat, with small stones or other material to fill the interstices.

Plastic limit. The moisture content at which the soil ceases to be in a plastic condition as determined by the plastic limit test.

Plasticity index. The numerical difference between the liquid limit and the plastic limit of a soil.

Pore water pressure. The pressure of the water in the voids of a saturated soil.

Porosity (Porosity ratio). The ratio of the volume of voids to the total volume of a material including voids.

Relative compaction. The percentage ratio of the dry density of the soil *in situ* to the maximum dry density of that soil as determined by the standard compaction test.

Sand. Small mineral particles from natural sources usually regarded as being of such a size that all will pass a No. 7 B.S. sieve, and free from appreciable amounts of clay and silt.

Sand fraction. In the particle-size analysis of soils mineral particles between the sizes 2·0 and 0·06 mm.

Saturated soil. A soil in which the voids are entirely filled with water.

Settlement. The downward movement of a soil or of the structure which it supports, resulting from a reduction in the voids in the underlying strata.

Silt. Mineral particles naturally deposited as sediment in water and usually regarded as of such a size that all will pass a 200 B.S. sieve, and free from appreciable amounts of clay.

Silt fraction. In the particle-size analysis of soils mineral particles between the sizes 0·06 and 0·002 mm.

Soil. Any naturally occurring loose or soft deposit forming part of the earth's crust and resulting from weathering or breakdown of rock formations or from the decay of vegetation.

Soil moisture suction. The reduced pressure (below atmospheric pressure), imparted to held water by the forces retaining the water above the water-table.

Soil profile. A vertical section showing the soil strata at a given site.

Stabilized soil. Soil treated in such a manner as to render its properties less affected by water or to increase its load-bearing capacity.

Sub-base. A layer of material placed between the foundation and the formation usually for a special purpose, such as a porous layer, or further to support the construction.

Subgrade. The natural foundation or the fill which directly receives the loads from the pavement.

Subsoil. The undisturbed strata lying immediately below the topsoil.

Surfacing. The course or courses above the base laid in the form of a continuous layer or layers to provide a wearing surface, to protect the base, or to add strength to the pavement.

Topsoil. The loose top layer of soil that can support vegetation.

Varves. Soils (usually clays) containing thin alternate layers of different particle sizes, formed from seasonal deposits from glacial streams.

Voids. The spaces in a material occupied by water, or air, or both water and air.

Voids ratio. The ratio of the volume of voids to the volume of solids in a material.

Water-table. The horizon in soil at which the pore water is at atmospheric pressure.

SYMBOLS

28·4

A = Area
a,r = Radius
B,b = Breadth
c = cohesion
CG = Centre of gravity
D = Depth
D_F = Depth factor
d = Diameter
d,x = Distance
E = Modulus of elasticity
e = Voids ratio
F = Factor of safety
f = Force of attraction
G_l = Specific gravity of liquid
G_s = Specific gravity of soil or stone particles
G_w = Specific gravity of water
g = Acceleration due to gravity
H = Relative humidity
H,h = Height, thickness
i = Hydraulic gradient; slope
K = Coefficient of saturated permeability
k = Modulus of subgrade reaction
LL = Liquid limit
L,l = Length
M = Molecular weight
m = Moisture content
P = Total load (wheel or point)
PI = Plasticity index
PL = Plastic limit
p = Pressure
p_s = Surcharge pressure
Q = Quantity of flow
q = Bearing pressure
q_a = Allowable bearing pressure
q_u = Ultimate bearing capacity
R = Universal gas constant
r = Gas constant per gram
r,a = Radius
s = Shear strength
T = Surface tension
t = Time; temperature
t_1 = Temperature difference
U = Percentage consolidation
u = Pore-water pressure
V = Volume
V_a = Air voids
v = Velocity
W,w = Weight
x,d = Distance
α = Contact angle
γ_b = Bulk density
γ_d = Dry density
γ_l = Density of liquid
γ_s = Density of particles
γ_w = Density of water
Δ = Displacement; any finite change
ε_t = Coefficient of thermal expansion
η = Viscosity of liquid
θ = Temperature (Absolute)
μ = Poisson's ratio
ρ = Settlement; penetration
σ = Stress
σ_e = Effective or intergranular stress
σ_n = Normal stress
σ_x, σ_y = Horizontal stress
σ_z = Vertical stress
τ = Shear stress
ϕ = Angle of shearing resistance
ψ = Capillary potential
ω = Angular velocity

ROCK

SEDIMENTARY

Conglomerate

Sandstone

Mudstone

Limestone or Chalk

Coal

Gypsum Rocksalt etc.

IGNEOUS

Plutonic (Granite)

Hypabyssal (Quartz porphyry)

Volcanic (Basalt)

Pyroclastic (Volcanic ash)

METAMORPHIC

Gneiss

Schist

Quartzite

Marble

Slate

Hard shale

SOILS

NON-COHESIVE

Gravel

Sand

Silt

Uncemented Calcareous matter (including shell beds)

COHESIVE

Clay

ORGANIC

Peat

COMPOSITE TYPES

Sandy gravel

Hoggin

Silty sand

Loam

Sandy clay

Silty clay

Boulder clay

Calcareous clay (Marl)

Stiff fissured clay

Weathered clay

Sandy peat

Silty peat

Clayey peat

Made ground

FIG. 28·1 SYMBOLS FOR ILLUSTRATING ROCK AND SOIL TYPES
(As recommended by Code of Practice for Site Investigations)

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